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Bridge Details and Specifications

103 Illustrations

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BRIDGE MEMBERS AND DETAILS
BRIDGE TABLES
BRIDGE SPECIFICATIONS
DESIGN OF PLATE GIRDERS

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PREFACE

The volumes of the International Library of Technology are made up of Instruction Papers, or Sections, comprising the various courses of instruction for students of the International Correspondence Schools. The original manuscripts are prepared by persons thoroughly qualified both technically and by experience to write with authority, and in many cases they are regularly employed elsewhere in practical work as experts. The manuscripts are then carefully edited to make them suitable for correspondence instruction. The Instruction Papers are written clearly and in the simplest language possible, so as to make them readily understood by all students. Necessary technical expressions are clearly explained when introduced.

The great majority of our students wish to prepare themselves for advancement in their vocations or to qualify for more congenial occupations. Usually they are employed and able to devote only a few hours a day to study. Therefore every effort must be made to give them practical and accurate information in clear and concise form and to make this information include all of the essentials but none of the non-essentials. To make the text clear, illustrations are used freely. These illustrations are especially made by our own Illustrating Department in order to adapt them fully to the requirements of the text.

In the table of contents that immediately follows are given the titles of the Sections included in this volume, and under each title are listed the main topics discussed.

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B

CONTENTS

NOTE.—This volume is made up of a number of separate Sections, the page numbers of which usually begin with 1. To enable the reader to distinguish between the different Sections, each one is designated by a number preceded by a Section mark (§), which appears at the top of each page, opposite the page number. In this list of contents, the Section number is given following the title of the Section, and under each title appears a full synopsis of the subjects treated. This table of contents will enable the reader to find readily any topic covered

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BRIDGE MEMBERS AND DETAILS

(PART 1)

MATERIALS

1. The materials most used in the construction of bridges are wood, iron, and steel. In this Section, the materials and the forms and shapes specially adapted to bridge work will be considered. The specifications governing the quality of materials, and the details of construction of bridges, will be treated in subsequent Sections.

WOOD

2. In the first bridge trusses that were built, wood was used almost wholly for compression members; to a great extent for tension members, such as lower chords; and for floor systems. As the loads at that time were light, and timber was plentiful in almost all localities, while iron and steel were expensive and of uncertain quality, wood was considered the most desirable material. With the advance of civilization, loads increased in weight, the clearing of large areas made timber scarce and expensive, and new methods of manufacture rendered iron and steel more reliable and less expensive. On this account, when the timber bridges first built deteriorated to such an extent that it became necessary to renew them, many were replaced by structures entirely of iron or steel, with the exception, in many cases, of the floor system, for which wood still remained the most satisfactory material.

sections between the members connected, and the resistance to crushing offered by the plates bearing on the rivets.

2. Friction of Riveted Joints.—As explained in *Bridge Members and Details*, Part 1, rivets are driven hot, and in cooling contract and hold firmly together the parts through which they pass. This causes a certain amount of friction between the parts, which helps to transmit stress from one member or part of a member to another. It is impossible to determine with any certainty how great this friction is, and it is customary to ignore it in the design of riveted joints, the shearing and bearing resistances alone being considered.

3. Shearing Value.—The maximum shearing stress that is allowable on a rivet is called the **shearing value** of the rivet, or the value of the rivet, in shear. If a rivet connects two plates, as represented in Fig. 1, the stress in one plate is transmitted to the other by means of the rivet, and the area subjected to shear is the area of cross-section of the rivet. As there is but one section c of the rivet subjected to shear, the rivet is said to be in **single shear**. When a rivet connects two members, as represented in Fig. 2, the rivet is subjected to shear at two sections d and e, and is said to be in **double shear**. In calculating the area of cross-section of a rivet, the nominal diameter is used.

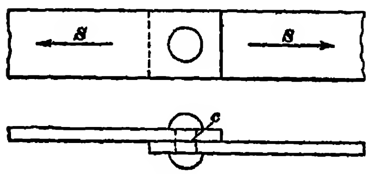


FIG 1

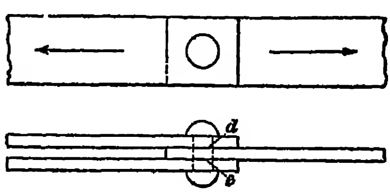


FIG 2

The shearing value of a rivet is found as follows:

Rule.—*To find the value of a rivet in single shear, multiply the area of cross-section of the rivet by the allowable intensity of stress in shear; to find the value of a rivet in double shear, multiply twice the area of cross-section of the rivet by the allowable intensity of stress in shear.*

bridge failures, however, soon led to the abandonment of this material for important members. For a number of years, it has been used only for unimportant parts, such as heavy bedplates or masonry plates under the ends of trusses, to distribute the pressure over the required area of the masonry. At the present time, it is being entirely superseded in bridge work by steel, even for unimportant details. It is a good rule *never to use cast iron for any part of a bridge the failure of which may cause damage to life or property.*

WROUGHT IRON

6. Properties.—Wrought iron is the product obtained by burning the carbon, and, to a certain extent, the other impurities from cast iron. The result is a porous mass of pure iron with some impurities; the smaller the quantity of these impurities, the better is the quality of the iron. The first product of the burning is hammered and pressed between rolls into a compact and homogeneous mass, to eliminate the pores and weld the iron firmly into a solid mass. The amount and method of hammering and rolling determine to a great extent the quality of the iron.

Wrought iron has a lower compressive strength than cast iron, and is more expensive; on the other hand, it possesses so many advantages over cast iron that it is far preferable in bridge work. By successive heating and hammering, it can be given almost any desired form; two pieces of it can readily be welded or riveted with very little injury to the iron; it is very tough and ductile, and gives ample warning before failure.

7. Use of Wrought Iron.—In the earlier bridge trusses, wrought iron was used for the tension web members, and later on replaced timber and cast iron in all the members, being used for several years with fairly good results. As the loads increased, it was necessary to build the bridges stronger; this could have been done by using heavier members of wrought iron, but, on account of the high cost of such members, it became very desirable to replace this

material by another having greater strength, and not inferior in other properties. Steel was found to possess the needed requirements, and has superseded wrought iron in nearly all forms of structural work.

8. The earlier grades of wrought iron were of comparatively low quality; the material lacked homogeneity, and possessed all the impurities of the cast iron from which it was made. With modern appliances for the manufacture of iron and steel, a much better quality can be obtained, and it is thought by some engineers that steel is used at present in some places where the conditions are such that wrought iron would give much better results. This is particularly true of highway bridges over railway tracks, where, on account of the sulphurous fumes from the smokestacks of locomotives, steel corrodes very rapidly. There are several instances where second-hand wrought-iron trusses supporting new steel floor systems over railway tracks were in excellent condition, so far as corrosion was concerned, after the steel floorbeams and stringers had corroded to such an extent that it was necessary to replace them. As a general rule, however, steel is far preferable to wrought iron for bridge work; wrought iron is used at present simply for members that require to be welded, and for details that cannot easily be manufactured of steel. The grades of steel used for bridge work do not weld well.

STEEL

9. Method of Manufacture.—Steel is manufactured in three ways; namely, by adding carbon to wrought iron, by removing carbon from cast iron, or by mixing cast and wrought iron in suitable proportions. The first process, which is called the **crucible process**, is the most expensive. Crucible steel is never used in bridge work.

The second process is known as the **Bessemer process**, and gives a grade of steel that is used to a great extent in structural work. Owing to the method of manufacture, Bessemer steel is liable to have hard and brittle spots, and

is not homogeneous. For this reason, it is not well adapted for use in members subject to shocks and blows, such as bridge members, although it is sometimes used.

The third process is known as the **open-hearth process,** and gives a grade of steel similar to Bessemer steel, but more homogeneous. Open-hearth steel is especially adapted to bridge work, and is largely used for that purpose; it is somewhat more expensive than Bessemer steel, but the additional expense is thought to be justified by the greater homogeneity obtained in the metal.

10. Red Shortness and Cold Shortness.—The strength and ductility of steel depend to a great extent on the amount of impurities, those usually present being silicon, manganese, carbon, sulphur, and phosphorus. The first two are sufficiently removed in modern methods of manufacture, and will not be further discussed. The amount of carbon can be regulated at will, according to the tenacity and ductility required, and varies from .1 to 1 per cent. of the product. In general, the lower the percentage of carbon, the softer is the steel; the higher the percentage of carbon, the harder is the steel.

The presence of sulphur makes the steel **red short;** that is, brittle and easily cracked when hot. This is a very undesirable property, as steel is heated and worked several times during the process of manufacture, and if much sulphur is present, the material is broken and wasted.

The presence of phosphorus makes steel **cold short;** that is, brittle and easily broken when cold. This, too, is a very undesirable property in steel that is to be used in bridge work, where the members are subject to shocks, and where the failure of a member may cause disastrous results.

For the reasons just stated, it would be desirable to remove sulphur and phosphorus entirely from steel; but, as the necessary process is too expensive, it is customary to require that the amounts of them shall not exceed a certain percentage of the product. The amount of sulphur allowed in steel for bridge work is usually not more than .04 per cent.; that of

phosphorus varies from .04 to .08 per cent., according to the method of removing the phosphorus.

11. Acid Process and Basic Process.—There are two methods of manufacture by both the Bessemer and the open-hearth process; these methods have been given the names **acid** and **basic**, the difference consisting in the lining of the receptacle in which the steel is prepared and the materials that are added during the process of preparation.

In the acid process, the receptacle is lined with silicious sandstone, and pig iron with a small percentage of phosphorus is used; none of the latter is removed by this method, the amount in the finished steel being the same as in the pig iron put in the receptacle.

In the basic process, the receptacle is lined with dolomite (magnesium limestone), and pig iron with a large amount of phosphorus may be used, as substances are added to the pig iron that remove most of the phosphorus.

It has been found in practice that, if steel is manufactured by the acid process, the amount of phosphorus should not exceed .08 per cent.; if made by the basic method, the phosphorus should not exceed .04 per cent.

12. Melts.—The quantity of steel that is manufactured in one operation of the melting furnace is spoken of as a **melt**. Each melt is numbered so that the record can be easily found by referring to the melt number. The chemical properties of each melt are placed on record for future reference as soon as the melt is finished.

13. When the proper chemical composition of the steel has been obtained, the molten metal in the receptacle (that is, the melt) is poured into molds, forming large castings called **ingots**. These ingots are sometimes rolled a few times to form what are known as **billets**. All the ingots or billets that come from one melt are stamped with the melt number. The billets are subsequently reheated and passed successively between rolls, each rolling decreasing the cross-section and increasing the length of the billet, until the desired form is reached. In this way, a great variety of shapes—such as

rods, plates, angles, channels, I beams, etc.—are obtained. The melt number on a billet should be stamped on each piece rolled from that billet.

The tenacity and ductility of the finished material are ascertained by certain mechanical tests prescribed by the bridge engineer in charge of the work and made under the supervision of inspectors appointed by him. The usual tests that are prescribed will be given in *Bridge Specifications.*

STRUCTURAL SHAPES OR SECTIONS

INTRODUCTION

14. Properties of Sections.—In *Strength of Materials*, the moment of inertia, radius of gyration, and section modulus have been defined, and the methods of computing them explained. In actual practice, it is seldom necessary to compute these properties for the simple rolled shapes, as they are computed by the steel manufacturers, who publish books containing tables in which the results of such computations are given. Several of these tables, and others that are convenient for bridge work, are given in *Bridge Tables.* All references in this Section are to those tables, unless otherwise stated.

15. Weight of Steel.—In *Bridge Tables*, the weight of a cubic foot of steel has been taken as 489.6 pounds. As the weight of 1 cubic foot of material is equal to that of 144 pieces 1 inch square and 12 inches long, the weight of each of these pieces of steel is $489.6 \div 144 = 3.4$ pounds. That is, *the weight of a piece of steel 1 inch square and 1 foot long is 3.4 pounds, and, therefore, the weight per linear foot of any piece of steel of uniform cross-section may be found by multiplying the area of the cross-section, in square inches, by 3.4.*

For example, the area of cross-section of a steel plate 12 inches wide and $\frac{1}{2}$ inch thick is 6 square inches; the weight of the plate, per foot, is, therefore, $6 \times 3.4 = 20.4$ pounds.

SIMPLE ROLLED SHAPES

SQUARE AND ROUND RODS

16. Areas and Weights.—Table I gives the areas of cross-section and weights per linear foot for round rods from $\frac{1}{16}$ inch to 6 inches in diameter; and Table II gives corresponding values for square rods from $\frac{1}{16}$ inch to 6 inches on a side. For example, it can be seen at a glance that the area of cross-section of a round rod $2\frac{7}{8}$ inches in diameter is 6.49 square inches, and that the weight per foot is 22.07 pounds. These rods are frequently used for tension members, such as lateral rods or counters, but are seldom used for the main members of trusses. The rods are sometimes connected at the ends by means of screw threads on which are turned nuts or other devices for transmitting to the rods the forces they are to resist.

17. Upset Screw Ends.—If screw threads are cut on the ends of a round rod, the area of cross-section at the root of the thread is considerably less than that of the body of the rod, and the screw ends are not so strong as the remainder of the rod. On this account, it is customary to enlarge the ends before the threads are cut; this is done by a process called **upsetting**. The rod, as originally made, is the same diameter throughout its entire length; in the process of upsetting, the ends are heated, and the diameter at the ends is enlarged to such an extent that, after the threads are cut on the enlarged ends, the area of cross-section at the root of the thread will be greater than that of the body of the bar; the greater area is necessary because the process of upsetting somewhat weakens the steel at the ends. When it is desired to form screw ends on square rods, the ends are upset to cylindrical forms. The rod is considerably shortened in upsetting, and so it is necessary to allow for the shortening effect.

Table III gives the dimensions of standard upset screw ends for round rods from $\frac{3}{4}$ inch to 3 inches in diameter, and

for square rods from $\frac{3}{4}$ inch to 3 inches on a side, together with the additional lengths of the original rods necessary to form a screw end. These dimensions give an area of cross-section at the root of the thread from 20 to 50 per cent. greater than that of the body of the bar, and have been found to be satisfactory in actual practice. It is seldom necessary in bridge work to upset a round rod greater than 3 inches in diameter, or a square rod greater than 3 inches on a side.

18. Hexagon Nuts.—Table IV gives the standard dimensions and weights of hexagon nuts, which are commonly used with rods with screw ends.

EXAMPLE.—A round rod $2\frac{5}{8}$ inches in diameter is required to be 10 feet long and to have two upset ends. (*a*) What is the diameter of the screw end? (*b*) What is the length of the screw end? (*c*) How long must the rod be before upsetting?

SOLUTION.—(*a*) Table III gives the diameter D of the screw end for a round rod $2\frac{5}{8}$ in. in diameter as 3 in. Ans.

(*b*) Table III gives the length L of the screw end for the same rod as 6 in. Ans.

(*c*) Table III gives the additional length U of rod necessary to form one upset end as $4\frac{3}{8}$ in. As there are two upset ends, the original length of the rod must be

$$10 \text{ ft.} + (2 \times 4\tfrac{3}{8} \text{ in.}) = 10 \text{ ft. } 8\tfrac{3}{4} \text{ in. Ans.}$$

FLAT PLATES

19. Areas and Weights.—Table VI gives the areas of cross-section, and Table VII gives the weights per linear foot, of steel plates from $\frac{1}{16}$ to 1 inch in thickness and from $\frac{1}{4}$ inch to 100 inches in width. The tables contain the usual sizes of plates; the areas and weights for other sizes can be found from these very readily by interpolation. Plates are used for webs and flanges of plate girders, and, in connection with other shapes, for tension and compression members in trusses.

20. Extreme Lengths of Plates.—Table V gives, approximately, the greatest lengths of various sizes of steel

plates that can be furnished by the rolling mills; these lengths should not be exceeded in designing bridges. In case plates are desired longer than the lengths given in the table, they can be formed by splicing together several pieces of shorter lengths, as explained in subsequent articles. In consulting Table V, if the width and thickness desired are not given in the table, it is necessary to use the length given for the next greater width and thickness.

21. Moments of Inertia of Rectangular Sections. Table VIII gives the values of the moments of inertia, about axes at right angles to the width, of rectangular sections from $\frac{1}{4}$ to 1 inch in thickness and from 2 to 60 inches in width. The value for any other thickness can be found very readily from those given in the table. As will be seen later, this table is extremely useful in calculating the properties of compound or built-up shapes.

EXAMPLE.—Assuming a steel plate 56 inches in width and $\frac{11}{16}$ inch in thickness: (*a*) what is the area of its cross-section? (*b*) what is its weight per linear foot? (*c*) what is the greatest length the rolling mills can furnish? (*d*) what is the value of its moment of inertia about an axis perpendicular to the width?

SOLUTION.—(*a*) As 56 is not given in the list of widths in Table VI, it will be well to consider the given plate equivalent to two plates, one of which is 50 in. and the other 6 in. in width. From Table VI, the area of cross-section of a plate 50 in. wide and $\frac{11}{16}$ in. thick is found to be 34.375 sq. in.; and the area of cross-section of a plate 6 in. wide and $\frac{11}{16}$ in. thick is found to be 4.125 sq. in. The sum of these, or 38.5 sq. in., is the area of cross-section of the given plate. Ans.

(*b*) Proceeding as in (*a*), we find, from Table VII, the weight per linear foot of a plate 6 in. in width and $\frac{11}{16}$ in. in thickness to be 14.03 lb., and that of a plate 50 in. in width and $\frac{11}{16}$ in. in thickness to be 116.9 lb. The sum of these, or 130.9 lb., is the weight per linear foot of the given plate. Ans.

(*c*) As, in Table V, 56 is not given in the list of widths, nor $\frac{11}{16}$ in the list of thicknesses, the next greater width, in this case 60 in., and the next greater thickness, in this case $\frac{3}{4}$ in., are looked for. The length corresponding to this width and thickness is 30 ft. Then, 30 ft. is approximately the longest $56'' \times \frac{11}{16}''$ plate that can be had.

(*d*) Consulting Table VIII, the value 10,061 for the moment of inertia is found opposite a width of 56 in. and below a thickness of $\frac{11}{16}$ in. Ans.

ANGLES

22. Properties.—Table IX gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of standard angles having equal legs; and Table X gives the same properties of angles having unequal legs. In bridge work, angles are employed more than any other single shape, with the exception of plates, and are used for flanges of plate girders, members of lateral trusses, and, in connection with other sections, for many built-up tension and compression truss members. The smaller sizes of angles, especially those having legs less than $2\frac{1}{2}$ inches in width, are never used except for unimportant details.

An angle is usually referred to by a product of three numbers, the first two of which express the widths of the legs, and the other the thickness. Thus, an $8'' \times 8'' \times \frac{3}{4}''$ angle is an angle each of whose legs is 8 inches wide and whose thickness is $\frac{3}{4}$ inch; a $6'' \times 4'' \times \frac{1}{2}''$ angle is an angle one of whose legs is 6 inches wide, the other 4 inches, and whose thickness is $\frac{1}{2}$ inch. In the case of unequal-leg angles, the width of the wider leg is usually written or named first, as in the example just given.

The symbol **L** is commonly used for *angle*, and **Ls** for *angles*. Thus, 2 **Ls** $6'' \times 4'' \times \frac{1}{2}''$ means two $6'' \times 4'' \times \frac{1}{2}''$ angles.

23. Actual Size and Nominal Size of Legs.—Angles are made by heating and rolling steel billets. The finishing process consists in passing the steel through special sets of rolls that give it the required angular form. Each one of these sets of rolls is used for angles of various thicknesses and of approximately the same width. In the construction of tables, these widths are considered to be all equal, and to have a common value equal to the actual width of the thinnest angle for which that set of rolls is employed. This width, for angles other than the thinnest, is called their **nominal width**, to distinguish it from their actual width. For the purposes of selecting angles in the general design

of members, the nominal width, which is smaller than the actual width, is used; the error is small, and on the side of safety. For detailing, making connections, etc., it is necessary to take into account the difference between actual and nominal widths.

24. Table XI gives the thicknesses of angles for which the actual width is equal to the nominal. Opposite a width of 5 in. × 3 in., for example, are found the thicknesses $\frac{5}{16}$ and $\frac{1}{2}$. This means that there are two sets of rolls for rolling 5″ × 3″ angles: the thinnest angle that is rolled with one of these sets is $\frac{5}{16}$ inch thick; the actual widths of the legs of those angles are 5 and 3 inches. The same set of rolls is used for angles whose thicknesses lie between $\frac{5}{16}$ inch and $\frac{1}{2}$ inch; but, although these angles are called 5″ × 3″ angles, they are a little wider than indicated by those figures. Likewise, the smallest thickness rolled with the other set of rolls is $\frac{1}{2}$ inch, for which the actual widths are 5 and 3 inches; for greater thicknesses, the actual widths are slightly greater.

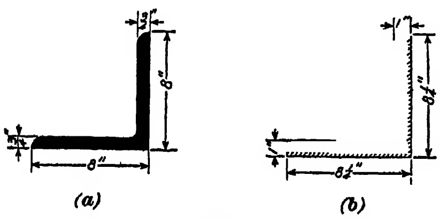


FIG. 1

Fig. 1 (*a*) shows a cross-section of an 8″ × 8″ × $\frac{3}{4}$″ angle. According to Table XI, these are actual dimensions, since the angle has the minimum thickness for which the rolls are used. For an 8″ × 8″ × 1″ angle, the same rolls are used, and are set $\frac{1}{4}$ inch farther apart to allow for the increase in thickness. The result is shown in Fig. 1 (*b*), the increase being shown by section lines. The actual size of this angle is, then, 8¼ in. × 8¼ in. × 1 in., although it is called an 8″ × 8″ × 1″ angle. In general, the actual width of an angle can be obtained from the nominal width by the following

Rule.—*Add to the nominal width the difference between the given thickness and the next smaller thickness for that width given in Table XI.*

For example, the actual size of a $6'' \times 4'' \times \frac{3}{4}''$ angle is $6\frac{3}{16}$ in. $\times$ $4\frac{3}{16}$ in. $\times$ $\frac{3}{4}$ in., since the next smaller thickness given in Table XI is $\frac{9}{16}$ inch, and the difference between $\frac{3}{4}$ and $\frac{9}{16}$ is $\frac{3}{16}$ inch.

25. Location of Gauge Lines.—Angles are connected to each other and to other shapes by rivets, the centers of which are located on lines, called **gauge lines**, parallel to the edges of the angles. In some cases, the rivets are located on one line, in others, on two, and sometimes, in 8-inch legs, on three lines. Table XII gives the standard distances from the gauge lines to the edges of the angles, together with the diameters of the largest rivets that can conveniently be driven into the leg.

CHANNELS

26. Channels are extensively used for chord members, compression web members, and columns for viaducts. Table XIII gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of channels varying in depth from 3 to 15 inches. The sizes given in bold-faced type are called **standard channels**, and are kept in stock by rolling mills; the others are rolled to order. Delay is sometimes caused in the work if any but standard channels are ordered, particularly when the rolling mills are busy.

27. Unlike angles, channels are not designated by the width and thickness, but by the depth and weight per linear foot. For instance, a 12-inch 25-pound (12″–25 #) channel is a channel 12 inches deep and weighing 25 pounds per linear foot.

The symbols ⊏ and ⊏s are used for *channel* and *channels*, respectively. For example, 3–12″–25 # ⊏s means three channels each 12 inches deep and each weighing 25 pounds per linear foot.

From Table XIII, any other dimension can be found when the depth and weight are given; thus, the area of cross-section of a 12-inch 25-pound channel is found in column 3 to be

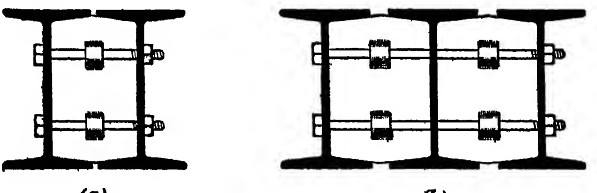
7.35 square inches; the thickness of web, .39 inch, is found in column 4; the width of flange, 3.05 inches, in column 5; etc. Columns 13 to 16 are very useful in working up connections when detailing. Column 17 gives the minimum distance between two channels of the same size, in order that the radius of gyration of the two acting together shall not be less than that given in column 8 for one channel alone.

I BEAMS

28. Dimensions and Properties.—I beams are extensively used to span small openings for both railroads and highways, for the stringers and floorbeams of highway bridges, and for columns supporting elevated railroads. Table XIV gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of I beams varying in depth from 3 to 24 inches. The sizes given in bold-faced type are called **standard I beams**, and are kept in stock by the rolling mills; the others are rolled to order.

29. I beams, like channels, are designated by the depth and weight per linear foot. For instance, a 20-inch 80-pound (20″–80 #) I beam is a beam 20 inches in depth and weighing 80 pounds per linear foot.

30. Cast-Iron Separators or Spacers.—When I beams are used for small spans under a railroad track, it is sometimes desirable to place two or three beams of the same



(a) (b)

FIG. 2

depth close together under each rail. In such a case, some device is necessary to make the beams work together and maintain a uniform distance apart. This is accomplished by

placing between the I beams cast-iron **spacers**, or **separators**, and bolting them in place with bolts passing through the spacers and beams. Fig. 2 (*a*) shows a cross-section of two I beams connected in this way, and Fig. 2 (*b*) shows three I beams connected.

The dimensions and weights of the usual forms of separators, or spacers, are given in Table XV.

OTHER SHAPES

31. Z Bars.—Z bars are used in connection with plates for columns, and in floors of bridges when solid steel floors are necessary. Table XVI gives the dimensions, areas of cross-section, weights per linear foot, and other useful properties of Z bars varying in depth from 3 to 6 inches. A Z bar is designated by giving all the widths and the thickness, the latter being written or named last; as a $3\frac{1}{4}'' \times 5'' \times 3\frac{1}{4}'' \times \frac{5}{16}''$ Z bar.

32. T Rails.—It is frequently desirable to know the properties of T rails, so that the strength can be determined in cases where they are subject to unusual stresses. Table XVIII gives the weights per yard, dimensions, and properties of American standard rail sections. It is customary to designate rails by the weight per linear yard; thus, a 90-pound rail is a rail that weighs 90 pounds per yard. When second-hand rails are used, proper allowance must be made for the decrease in section due to wear and rust.

REDUCTION OF INCHES TO DECIMALS OF 1 FOOT OR INCH

33. Table XVII is very useful for converting inches into decimal fractions of 1 foot, and for converting into decimal fractions the usual fractional divisions of 1 inch. The last column on the right contains the decimal equivalents of the common fractions in the column immediately preceding it. The numbers in the extreme left column are fractions of an inch; those at the top of the other columns represent

whole inches; and those in the body of the table, decimals of a foot, each of those decimals corresponding to the number of inches denoted by the number at the top of the column, plus the fraction in the left-hand column, horizontally opposite the decimal in question. Thus, the decimal of a foot corresponding to $10\frac{5}{16}$ inches is .8594, found in the column headed 10, and horizontally opposite the fraction $\frac{5}{16}$ in the first column.

EXAMPLES FOR PRACTICE

1. What is the weight of a piece of steel 8.5 feet long, if the area of cross-section is 6.7 square inches? Ans. 193.63 lb.

2. A square steel rod $1\frac{7}{8}$ inches on a side is required to be 15 feet long and to have two upset screw ends. Find, from the tables: (*a*) the diameter of the upset end; (*b*) the length of rod before upsetting; (*c*) the weight of two hexagon nuts for this rod.

Ans. $\begin{cases} (a) & 2\frac{3}{4} \text{ in.} \\ (b) & 15 \text{ ft. } 10\frac{1}{4} \text{ in.} \\ (c) & 19.60 \text{ lb.} \end{cases}$

3. A steel plate has a width of 45 inches and a thickness of $\frac{9}{16}$ inch. Find, from the tables: (*a*) the area of cross-section of the plate; (*b*) the weight per linear foot; (*c*) the greatest length the rolling mills can furnish.

Ans. $\begin{cases} (a) & 25.31 \text{ sq. in.} \\ (b) & 86.06 \text{ lb.} \\ (c) & 40 \text{ ft.} \end{cases}$

4. Find, from the tables, the actual dimensions of angles the nominal dimensions of which are as follows: (*a*) $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{7}{16}$ in.; (*b*) 6 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in.; (*c*) 6 in. $\times$ 6 in. $\times$ $\frac{15}{16}$ in.; (*d*) 7 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{11}{16}$ in.

Ans. $\begin{cases} (a) & 3\frac{5}{8} \text{ in.} \times 3\frac{5}{8} \text{ in.} \times \frac{7}{16} \text{ in.} \\ (b) & 6\frac{1}{8} \text{ in.} \times 3\frac{5}{8} \text{ in.} \times \frac{1}{2} \text{ in.} \\ (c) & 6\frac{1}{4} \text{ in.} \times 6\frac{1}{4} \text{ in.} \times \frac{15}{16} \text{ in.} \\ (d) & 7\frac{3}{16} \text{ in.} \times 3\frac{11}{16} \text{ in.} \times \frac{11}{16} \text{ in.} \end{cases}$

5. Find, from the tables, the decimal parts of a foot equivalent to: (*a*) $1\frac{1}{2}$ inches; (*b*) $2\frac{3}{4}$ inches; (*c*) $5\frac{7}{8}$ inches; (*d*) $11\frac{15}{32}$ inches.

Ans. $\begin{cases} (a) & .1250 \\ (b) & .2292 \\ (c) & .4896 \\ (d) & .9557 \end{cases}$

6. Find, from the tables, the decimal parts of an inch equivalent to: (*a*) $\frac{7}{32}$ inch; (*b*) $1\frac{9}{16}$ inches; (*c*) $2\frac{29}{32}$ inches.

Ans. $\begin{cases} (a) & .2188 \\ (b) & 1.5625 \\ (c) & 2.9063 \end{cases}$

RIVETS

34. Introductory.—The rolled shapes that have been described in the preceding articles are connected to one another and to other shapes by means of rivets, those most commonly used in bridge work varying from $\frac{1}{2}$ to 1 inch in diameter. Circular holes $\frac{1}{16}$ inch larger in diameter than the rivets are first punched or drilled in the shapes that are to be connected; the shapes are then placed on one another so that the holes come in line; and heated rivets are inserted in the holes. As soon as a rivet is inserted, and before it has time to cool, the protruding end is hammered or pressed until the rivet completely fills the hole, when a head is formed by tools specially adapted to that purpose.

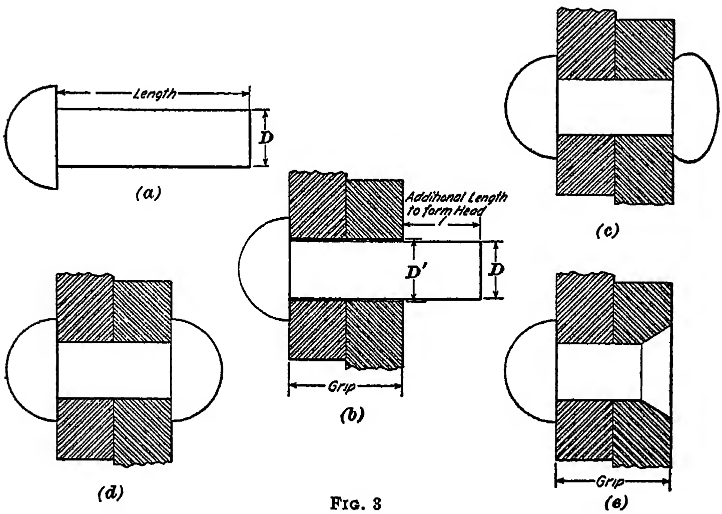


FIG. 3

35. Full and Countersunk Heads.—Each rivet, before driving, has one head, as represented in Fig. 3 (*a*). In Fig. 3 (*b*) is shown a rivet inserted in the hole and ready for driving; Fig. 3 (*c*) shows the rivet when partly driven; and Fig. 3 (*d*) shows it when completely driven and finished, with the other head formed. This style of rivet is called a

button-head rivet. The rivet represented in Fig. 3 (*d*) is said to have two **full heads.**

When it is objectionable, for any reason, to have the head protrude beyond the surface of the metal, one or both ends of the hole are enlarged to a conical form, and a shorter rivet is used, which is made just long enough to fill the enlarged hole, as represented at the right end of the rivet in Fig. 3 (*e*). Owing to the difficulties of manufacture, such a head will usually project about $\frac{1}{8}$ inch beyond the surface of the metal. This form of head is called a **countersunk head,** and the rivet, a **countersunk rivet.** It is sometimes desirable to use two countersunk heads, one at each end of the rivet.

36. Dimensions of Heads.—Table XIX gives the usual dimensions of button and countersunk heads for rivets from $\frac{1}{2}$ to 1 inch in diameter. In case the heads are too high, and it is not desired to use countersunk heads, they may be flattened by hammering when hot to a height of $\frac{3}{8}$, $\frac{1}{4}$, or $\frac{1}{8}$ inch. If it is desired at any point that a countersunk head should not project at all beyond the surface of the metal, it must be planed or chipped off with a chisel.

37. Dimensions and Weights.—The distance from the under side of the head to the end of the rivet before driving, as represented in Fig. 3 (*a*), is called the **length;** the cylindrical portion of the rivet is called the **shank;** and the thickness of metal between the heads, as represented in Fig. 3 (*b*), is called the **grip.** Table XX gives the additional length of shank required to form a head for rivets from $\frac{1}{2}$ to 1 inch in diameter and for grips of from $\frac{1}{2}$ inch to 6 inches. Part of this additional length is utilized in filling the rivet hole whose diameter is D', Fig. 3 (*b*), and the remainder in forming the head. On account of the necessity of filling the hole, a greater additional length is required for long grips than for short grips.

To obtain the length of rivet required for a given grip and diameter of rivet, add to the grip the additional length given in Table XX for that rivet corresponding to that grip.

For example, if a $\frac{7}{8}$-inch rivet has a grip of $3\frac{1}{2}$ inches, then, to form a button head, since the additional length corresponding to a grip of $3\frac{1}{2}$ inches for a $\frac{7}{8}$-inch rivet is given in Table XX as $1\frac{3}{4}$ inches, the length before driving should be $3\frac{1}{2} + 1\frac{3}{4} = 5\frac{1}{4}$ inches, and to form a countersunk head, $3\frac{1}{2} + 1 = 4\frac{1}{2}$ inches.

The weights of button heads for rivets from $\frac{1}{2}$ to 1 inch in diameter are given in Table XXI. The weights of shanks can be found by the use of Table I.

EXAMPLE.—To find the weight of 125 rivets $\frac{7}{8}$ inch in diameter, having a grip of $2\frac{5}{8}$ inches, if both ends are to have button heads.

SOLUTION.—The additional length of shank required to form a button head on a $\frac{7}{8}$-in. rivet having a grip of $2\frac{5}{8}$ in. is given in Table XX as $1\frac{5}{8}$ in.; then, the length before driving will be $2\frac{5}{8} + 1\frac{5}{8} = 4\frac{1}{4}$ in., or, by Table XVII, .3542 ft. Table I gives the weight per linear foot of a round rod $\frac{7}{8}$ in. in diameter as 2.04 lb.; then, the weight of the rivet shanks, since there are 125 rivets, is

$$125 \times .3542 \times 2.04 = 90.32 \text{ lb.}$$

Table XXI gives the weight of 100 button heads for $\frac{7}{8}$-in. rivets as 16.7 lb.; then, the weight of 125 heads is $\frac{125}{100} \times 16.7 = 20.88$ lb. The total weight of the rivets is

$$90.32 + 20.88 = 111.20 \text{ lb. Ans.}$$

38. Conventional Signs for Riveting.—Those rivets that are driven where the bridge is manufactured are called **shop rivets**; those that are driven where the bridge is erected, **field rivets**. Table XXIII (*a*) and (*b*) shows the conventional signs used in preparing drawings to indicate where the rivet is to be driven and what type of head is desired. By **this side, outside,** or **near side** is meant the surface shown uppermost on the drawing, or the upper side; by **other side, inside,** or **far side** is meant the surface opposite that shown uppermost, or the under side. Both of these systems are standard in different offices; that shown in Table XXIII (*a*) is the **American Bridge Company's standard**; that shown in Table XXIII (*b*) is called the **Osborne standard.**

39. Rivet Pitch.—The distance between the centers of two consecutive rivets in the same row is called **the pitch**

of the rivets. If there is one row, as in Fig. 4 (*a*), the pitch is the distance between two rivets in the same row; if there are two rows, as represented in Fig. 4 (*b*), the pitch is the distance between two consecutive rivets in alternate rows, measured parallel to the gauge lines. The pitch is marked *p* in Fig. 4.

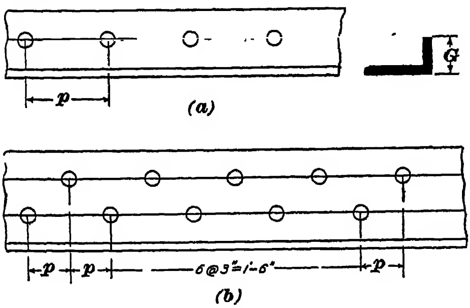


FIG. 4

In spacing rivets along gauge lines, it is customary to make the pitch uniform, that is, to have as many spaces of the same length together as is convenient. In specifying the number of spaces, pitch, etc., it is customary to write first the number of equal spaces, then the pitch, and finally the distance covered by those equal spaces [see Fig. 4 (*b*)].

Table XXII gives the total distances covered by equal spaces from $1\frac{1}{2}$ to 6 inches, and from 2 to 32 in number.

EXAMPLES FOR PRACTICE

1. Find the total length of shank required to form a button head for: (*a*) a $\frac{3}{4}$-inch rivet having a grip of 2 inches; (*b*) a $\frac{7}{8}$-inch rivet having a grip of $3\frac{1}{8}$ inches; (*c*) a 1-inch rivet having a grip of $3\frac{1}{2}$ inches.

Ans. $\begin{cases} (a)\ 3\frac{1}{2} \text{ in.} \\ (b)\ 4\frac{7}{8} \text{ in.} \\ (c)\ 5\frac{3}{8} \text{ in.} \end{cases}$

2. Find the total length of shank required to form one countersunk head for: (*a*) a $\frac{3}{4}$-inch rivet having a grip of $2\frac{1}{4}$ inches; (*b*) a $\frac{7}{8}$-inch rivet having a grip of $2\frac{3}{4}$ inches.

Ans. $\begin{cases} (a)\ 3\frac{1}{8} \text{ in.} \\ (b)\ 3\frac{5}{8} \text{ in.} \end{cases}$

3. Find the weight of 245 rivets $\frac{3}{4}$ inch in diameter and having a grip of $2\frac{1}{8}$ inches: (*a*) if both ends have button heads; (*b*) if one end has a countersunk head.

Ans. $\begin{cases} (a)\ 141 \text{ lb.} \\ (b)\ 122 \text{ lb.} \end{cases}$

4. Find, from Table XXII, the total distances covered by the following numbers of equal spaces: (*a*) eleven spaces at $2\frac{7}{8}$ inches;

(*b*) seventeen spaces at $3\frac{3}{8}$ inches; (*c*) twenty spaces at $3\frac{1}{8}$ inches; (*d*) twenty-seven spaces at $4\frac{3}{4}$ inches; (*e*) thirty-one spaces at $3\frac{1}{2}$ inches

Ans. (*a*) 2 ft. $7\frac{5}{8}$ in.
(*b*) 4 ft. $9\frac{3}{8}$ in.
(*c*) 5 ft. $2\frac{1}{2}$ in.
(*d*) 10 ft. $8\frac{1}{4}$ in.
(*e*) 9 ft. $\frac{1}{2}$ in.

BUILT-UP SHAPES

40. The simple shapes that have been explained and tabulated in the preceding pages are seldom used alone for bridge members; they are usually employed in connection with other shapes, the several parts being thoroughly connected and riveted together so as to act as a single shape. Such combinations are called **built-up,** or **compound, shapes,** the same names being applied to members formed with them. The methods of connecting the different parts of a built-up shape will be treated later.

TWO ANGLES BACK TO BACK

41. Angles are frequently used in pairs, the backs being placed either together or parallel to each other and a short distance apart. When two angles are used for a compression member, it is necessary to know the radii of gyration of the shape about two axes passing through the center of gravity, one at right angles, and the other parallel to the adjacent backs of the angles. Tables XXIV, XXV, and XXVI give the radii of gyration about the two axes passing through the center of gravity of the section, for pairs of angles when placed in contact and also at distances of $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{3}{4}$ inch apart, respectively. Only values for the maximum and minimum thicknesses of angles are given; those for any intermediate thickness can be found by interpolation.

EXAMPLE.—To find, from Table XXIV, the radius of gyration about an axis parallel to the adjacent backs of two $6'' \times 6'' \times \frac{5}{8}''$ angles placed $\frac{1}{4}$ inch apart.

SOLUTION.—It is found, from the table, that the required radius of gyration for two $6'' \times 6'' \times \frac{3}{8}''$ angles is 2.58 in., and for two $6'' \times 6'' \times 1''$

angles, 2.68 in. The difference in the thickness of these angles is $\frac{5}{8}$, or $\frac{10}{16}$, in., and the difference between the radii of gyration is $2.68 - 2.58 = .10$ in., or $\frac{.10}{10} = .01$ for each $\frac{1}{16}$ inch increase in the thickness of the angle. The difference between the minimum thickness, $\frac{3}{8}$ inch, and that of the angles that are to be used, $\frac{5}{8}$ inch, is $\frac{4}{16}$; then, the increase in the radius of gyration is $.01 \times 4 = .04$, and the required radius is

$$2.58 + .04 = 2.62 \text{ in.} \quad \text{Ans.}$$

OTHER BUILT-UP SHAPES

42. Standard Forms.—Fig. 5 shows the forms of built-up shapes in most common use. Forms (*a*) to (*e*) are composed of plates, angles, and channels, arranged as shown, and are used for upper chords and end posts. The vertical plates are called the **web-plates**; the angles that connect to their edges, the **flanges**; and the horizontal plate at the top, the **cover-plate.**

Forms (*f*) to (*l*) are composed of simple shapes, arranged symmetrically as shown, and are used for lower chords and web members of trusses, and for columns in trestle bents. Forms (*m*) and (*n*) are composed of channels and **I** beams arranged symmetrically, and are used, to some extent, for columns for elevated railways.

43. Properties.—When a built-up shape is used for a compression member, it is necessary to know the area of cross-section, and also the radii of gyration about two axes passing through the center of gravity of the cross-section, one of which is parallel and the other at right angles to the webs. The area of cross-section can be readily found by adding together the areas of cross-section of the different parts of which the shape is composed. The location of the center of gravity can be determined by the principles of statics, as explained in *Analytic Statics*, Part 2, and the radii of gyration can be found as explained in *Strength of Materials*, Part 2. There is so great a variety of built-up shapes that it is difficult to prepare tables covering all the cases that are likely to occur in practice. In any particular case, the required quantities must be determined by calculation.

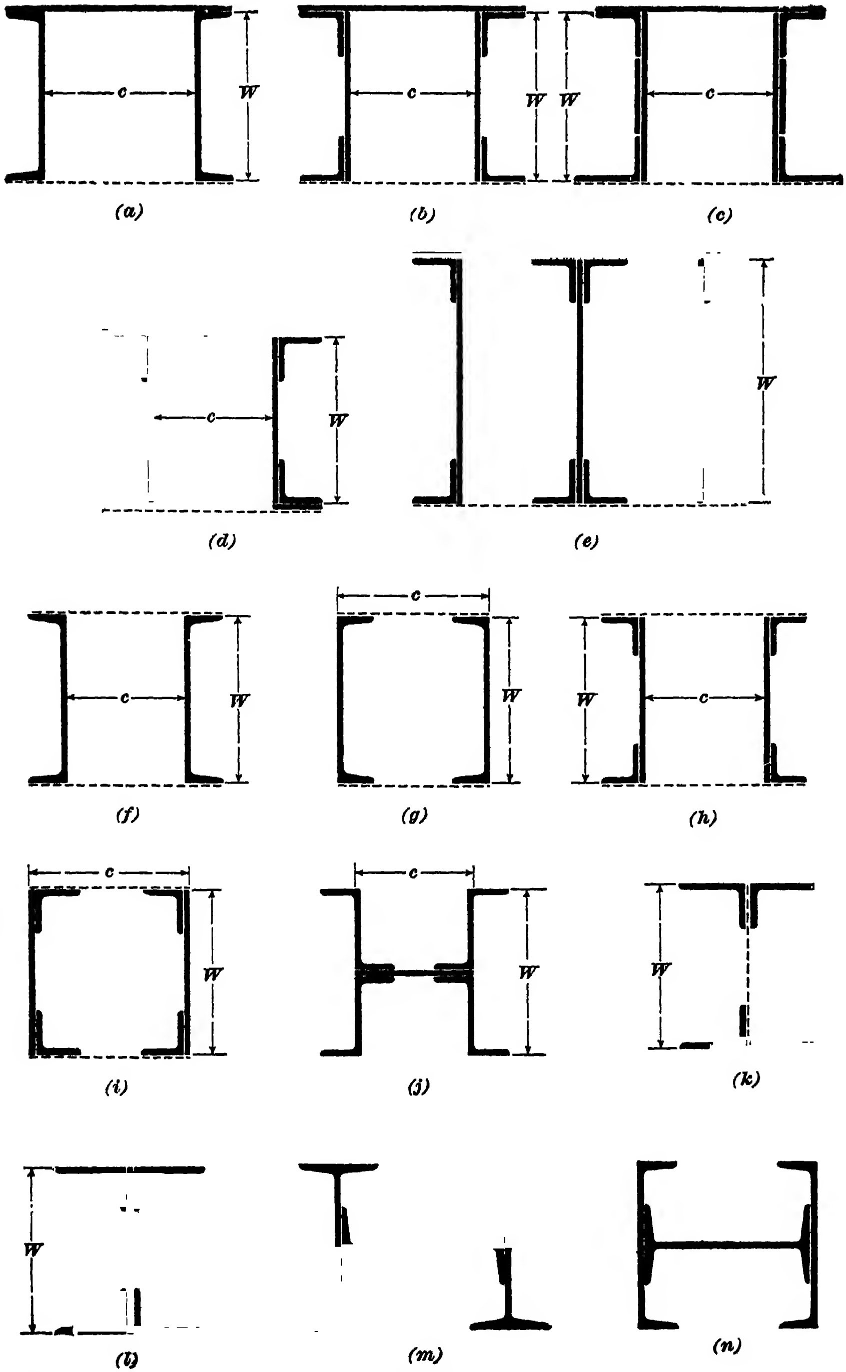


Fig. 5

44. In order to find the radius of gyration of a section with reference to any axis, it is necessary to know the moment of inertia of the section about that axis. As explained in *Strength of Materials*, Part 2, the moment of inertia of a plane surface about an axis in the same plane and not passing through the center of gravity of the surface is equal to the sum of the moment of inertia of the surface about an axis through its center of gravity parallel to the given axis, and the product of the area of the surface and the square of the perpendicular distance from its center of gravity to the given axis. If I_1 represents the moment of inertia of a surface about an axis not passing through its center of gravity, I_2 its moment of inertia about an axis through its center of gravity and parallel to the first axis, A_1 the area of the surface, and y_1 the perpendicular distance from the center of gravity of the surface to the first axis, then,

$$I_1 = I_2 + A_1 y_1^2$$

If I represents the moment of inertia of a surface composed of several smaller surfaces, and I_1', I_1'', I_1''', etc. represent the moments of inertia of the smaller surfaces about the same axis, then

$$I = I_1' + I_1'' + I_1''' + \ldots$$

or, letting ΣI_1 represent the sum $I_1' + I_1'' + I_1''' + \ldots$

$$I = \Sigma I_1 \qquad (1)$$

Also, substituting for I_1 its value $I_2 + A_1 y_1^2$,

$$I = \Sigma\,(I_2 + A_1 y_1^2) \qquad (2)$$

EXAMPLE 1.—A built-up section, arranged as shown in Fig. 6, is composed of the following simple sections: one cover-plate 26 in. × $\frac{1}{2}$ in.; two upper flange angles 4 in. × 4 in. × $\frac{5}{8}$ in.; two web-plates 20 in. × $\frac{5}{8}$ in.; and two lower flange angles 6 in. × 4 in. × $\frac{5}{8}$ in. To find: (*a*) the area of the section, and the location of the two axes $Y'Y$, parallel, and $X'X$, perpendicular, to the webs, and passing through the center of gravity of the section; (*b*) the moment of inertia about each of these axes; (*c*) the radius of gyration referred to each of these axes.

NOTE.—The vertical distance from the top surface of the upper flange angle to the bottom surface of the lower flange angle, 20¼ inches in Fig. 6, is called the **distance back to back of angles**, and is made ¼ inch greater than the width of the web-plate, to allow for irregularities in the latter. The horizontal distance between the web-plates, 15 inches in Fig. 6, is called the **clear distance between webs.**

SOLUTION.—(*a*) As the section is symmetrical about the axis $Y'Y$, half way between the webs, the center of gravity lies on this axis. As the section is not symmetrical about any axis at right angles to $Y'Y$, the location of the axis $X'X$, perpendicular to $Y'Y$, must be found by calculation. The areas of cross-section and the location of centers of gravity of the simple sections will be taken from the tables. As

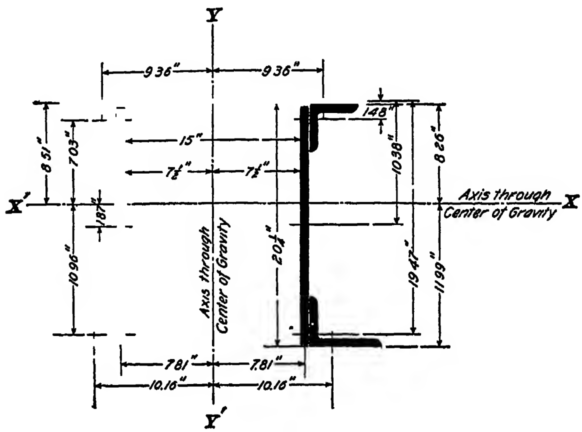


FIG. 6

moments may be taken about any axis, it will be convenient to take them about an axis passing through the center of the cover-plate. The lever arms are shown at the right-hand side in Fig. 6. For example, the distance from the back of a $4'' \times 4'' \times \frac{5}{8}''$ angle to the center of gravity is given in Table IX as 1.23 in.; then, its lever arm is equal to the sum of 1.23 and one-half the thickness of the cover-plate, or $1.23 + .25 = 1.48$ in.; similarly for the other simple sections. The calculation is as follows:

SECTIONS	AREA FROM TABLES SQUARE INCHES	LEVER ARM	MOMENTS
One plate 26 in. $\times \frac{1}{2}$ in.	$= 13.00$	0	0
Two angles 4 in. $\times$ 4 in. $\times \frac{5}{8}$ in.	$2 \times 4.61 = 9.22$	1.48	13.65
Two plates 20 in. $\times \frac{5}{8}$ in.	$2 \times 12.5 = 25.00$	10.38	259.50
Two angles 6 in. $\times$ 4 in. $\times \frac{5}{8}$ in.	$2 \times 5.86 = 11.72$	19.47	228.19
Total,	58.94		501.34

Dividing the total moment (501.34) by the entire area (58.94) of the section gives $501.34 \div 58.94 = 8.51$ in. from the center of the cover-plate to the axis $X'X$. Then, $8.51 - .25 = 8.26$ in. is the distance from the axis $X'X$ to the back of the upper flange angles. Ans.

(*b*) The moment of inertia about the axis $Y'Y$ will first be found The lever arms y_1 are plainly shown in Fig. 6.

	ONE PLATE 26 IN. $\times \frac{1}{2}$ IN.	
I_1 (Table VIII)	=	7323
$A_1 y_1^2 = 13 \times 0 \times 0$	=	0
	TWO ANGLES 4 IN. $\times$ 4 IN. $\times \frac{5}{8}$ IN.	
I_2 (Table IX) $= 2 \times 6.66$	=	13.3
$A_1 y_1^2 = 9\ 22 \times 9.36^2$	=	807.8
	TWO PLATES 20 IN. $\times \frac{5}{8}$ IN.	
$I_1 = 2 \times \frac{1}{12} \times 20 \times (\frac{5}{8})^3$	=	.8
$A_1 y^2 = 25 \times 7\ 81^2$	=	1524.9
	TWO ANGLES 6 IN. $\times$ 4 IN. $\times \frac{5}{8}$ IN.	
I_1 (Table X) $= 2 \times 21\ 07$	=	42.1
$A_1 y^2 = 11\ 72 \times 10\ 16^2$	=	1209.8
Moment of inertia about $Y'Y$	=	4331.0 Ans.

The moment of inertia about the axis $X'X$ will now be found. The lever arms y_1 are shown at the left-hand side in Fig. 6.

	ONE PLATE 26 IN. $\times \frac{1}{2}$ IN.	
$I_1 = \frac{1}{12} \times 26 \times (\frac{1}{2})^3$	=	.3
$A_1 y_1^2 = 13 \times 8.51^2$	=	941.5
	TWO ANGLES 4 IN. $\times$ 4 IN. $\times \frac{5}{8}$ IN.	
I_1 (Table IX) $= 2 \times 6.66$	=	13.3
$A_1 y_1^2 = 9.22 \times 7.03^2$	=	455.7
	TWO PLATES 20 IN. $\times \frac{5}{8}$ IN.	
I_1 (Table VIII) $= 2 \times 416.67$	=	833.3
$A_1 y_1^2 = 25\ 0 \times 1\ 87^2$	=	87.4
	TWO ANGLES 6 IN. $\times$ 4 IN. $\times \frac{5}{8}$ IN.	
I_1 (Table X) $= 2 \times 7.52$	=	15.0
$A_1 y_1^2 = 11\ 72 \times 10\ 96^2$	=	1407.8
Moment of inertia about $X'X$	=	3754.8 Ans.

(*c*) The radius of gyration is found by the formula $r = \sqrt{\frac{I}{A}}$. If r_y and r_x are the radii of gyration about $Y'Y$ and $X'X$, respectively, this formula gives

$$r_y = \sqrt{\frac{4{,}331.0}{58\ 94}} = \sqrt{73\ 48} = 8.57 \text{ in. Ans.}$$

$$r_x = \sqrt{\frac{3{,}754\ 8}{58\ 94}} = \sqrt{63.70} = 7.98 \text{ in. Ans.}$$

EXAMPLE 2.—A built-up section is composed of two web-plates 15 in. $\times \frac{1}{2}$ in , and four flange angles 4 in $\times$ 4 in. $\times \frac{1}{2}$ in , arranged as shown in Fig. 7. To find: (*a*) the area of the entire cross-section;

(*b*) the location of the axes passing through the center of gravity, (*c*) the moment of inertia with respect to each axis; (*d*) the radius of gyration with respect to each axis.

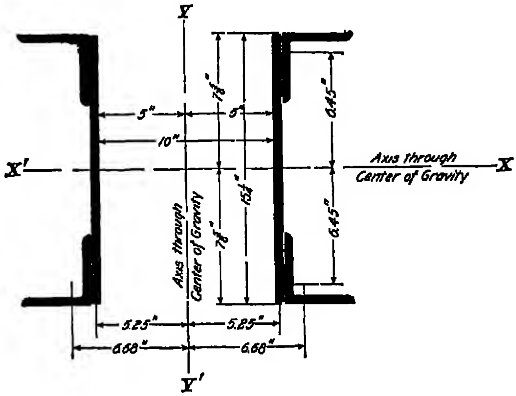


FIG 7

SOLUTION —(*a*) The area of two plates 15 in. $\times \frac{1}{2}$ in is 15 sq in.; the area of four angles 4 in $\times$ 4 in $\times \frac{1}{2}$ in is $4 \times 3.75 = 15$ sq in. Then, the area of the entire section is

$$15 + 15 = 30 \text{ sq. in.} \quad \text{Ans.}$$

(*b*) As the section is symmetrical, the axis $Y' Y$ is 5 in from the inside surface of each web, and the axis $X' X$ is $7\frac{5}{8}$ in. from the backs of the flange angles. Ans

(*c*) The distances from $X' X$ to the center of gravity of the various angles is shown at the right in Fig. 7.

TWO PLATES 15 IN. $\times \frac{1}{2}$ IN.

I_s (Table VIII) $= 2 \times 140\ 63$ . . . . $= 281.3$

FOUR ANGLES 4 IN. $\times$ 4 IN. $\times \frac{1}{2}$ IN

I_s (Table IX) $= 4 \times 5\ 56$ . . . . $= 22\ 2$

$A_1 y_1^2 = 15 \times 6.45^2$ . . . . $= 624.0$

Moment of inertia about $X' X$ . . . . $= 927\ 5$ Ans

The distance from $Y' Y$ to the center of gravity of the angles and webs is shown on the lower side in Fig. 7.

TWO PLATES 15 IN. $\times \frac{1}{2}$ IN.

$I_s = \frac{2}{12} \times 15 \times (\frac{1}{2})^3$ . . . . $=$ .3

$A_1 y_1^2 = 15 \times 5.25^2$ . . . . $= 413.4$

FOUR ANGLES 4 IN $\times$ 4 IN. $\times \frac{1}{2}$ IN.

I_s (Table IX) $= 4 \times 5\ 56$ . . . . $= 22\ 2$

$A_1 y_1^2 = 15 \times 6\ 68^2$ . . . . $= 669.3$

Moment of inertia about $Y' Y$ . . . . $= 1105.2$ Ans

(*d*) $r_x = \sqrt{\frac{I}{A}} = \sqrt{\frac{927.5}{30}} = \sqrt{30\ 92} = 5.55$ in. Ans.

$r_y = \sqrt{\frac{1,105.3}{30}} = \sqrt{36\ 84} = 6\ 07$ in. Ans.

EXAMPLE 3.—A built-up section is composed of four $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ angles arranged as shown in Fig. 8. To find: (*a*) the area of the cross-section, (*b*) the moments of inertia with respect to axes through the center of gravity, (*c*) the radii of gyration with respect to the same axes.

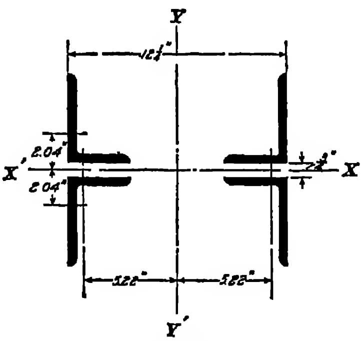


FIG 8

SOLUTION —(*a*) The area of the cross-section is equal to

$4 \times 4 = 16$ sq. in. (Table X). Ans.

(*b*) As the section is symmetrical, its center of gravity is its center of symmetry.

The moment of inertia with respect to the axis $X'X$ is as follows:

FOUR ANGLES 5 IN. × 3½ IN. × ½ IN.

$I_a = 4 \times 9.99$	=	39 96
$A_1 y_1^2 = 16 \times 2.04^2$	=	66.59
		106.55 Ans.

The moment of inertia with respect to the axis $Y'Y$ is as follows:

FOUR ANGLES 5 IN. × 3½ IN. × ½ IN.

$I_a = 4 \times 4.05$	=	16 20
$A_1 y_1^2 = 16 \times 5.22^2$	=	435.97
		452 17 Ans.

(*c*) $r_x = \sqrt{\frac{106.55}{16}} = \sqrt{6.66} = 2.58$ in. Ans.

$r_y = \sqrt{\frac{452\ 17}{16}} = \sqrt{28.26} = 5.32$ in. Ans.

The radius of gyration with respect to the axis $X'X$ could in this example have been found from Table XXVI, as the radius of gyration of the four angles with respect to this axis is the same as that of two angles. In Table XXVI, the radius of gyration for two angles with the short legs parallel and $\frac{3}{4}$ in. apart is given in the column headed r_4; that for two angles 5 in. × $3\frac{1}{2}$ in × $\frac{5}{16}$ in is 2.54 in.; and that for two angles 5 in. × $3\frac{1}{2}$ in × $\frac{7}{8}$ in. is 2 65 in. The difference in thickness of these angles is $\frac{9}{16}$ in , and the difference between their radii of gyration is .11. The difference in thickness between $\frac{5}{16}$ and $\frac{1}{2}$ is $\frac{3}{16}$;

then the difference between the radii of gyration is [illegible] .11 = .04, which added to 2 54 gives 2 58 in , as before.

45. Remarks on Example 1.—In example 1 of Art. 44, it will be seen that the radius of gyration r_y with respect to $Y'Y$ is greater than the radius r_x with respect to the axis $X'X$; also, that the clear distance between webs (c, Fig. 5), is approximately equal to three-fourths of the distance W back to back of the flange angles. It can be shown that, for the sections (a), (b), (c), and (d), Fig. 5, r_y is invariably greater than r_x when c is greater than $\frac{3}{4}W$. Therefore, if, as is usually the case, only the least radius of gyration is required, it is unnecessary to compute r_y if c is greater than $\frac{3}{4}W$. This relation does not hold for section (e), for which both radii of gyration must be computed in order to find the smaller. It will also be seen that r_x is approximately equal to $\frac{3}{8}W$, and it may be shown that this relation is true in almost all cases for the forms (a), (b), (c), (d), and (e). This is convenient in approximate calculations, and is made use of in designing before the final cross-section of a member has been found.

46. Remarks on Example 2.—In example 2 of Art. 44, it will be seen that r_y is greater than r_x, and that c is approximately equal to $\frac{2}{3}W$. It may be shown that, if only the least radius of gyration is required, it is unnecessary to compute r_y if c is greater than $\frac{2}{3}W$, for the forms (f) and (h), Fig 5, and, if c is greater than $\frac{7}{8}W$, for the forms (g) and (i) For approximate calculation, the least radius of gyration for the forms (f), (g), (h), and (i) will be r_x, and may be taken equal to $\frac{3}{8}W$ if c and W bear the proper relation, that is, if c is greater than $\frac{2}{3}W$, for the forms (f) and (h), and greater than $\frac{7}{8}W$ for the forms (g) and (i).

47. It is usually specified that the ratio $\frac{l}{r}$ of the unsupported length l of a compression member to its least radius of gyration r shall not exceed a certain number, generally from 80 to 120. The approximate relations stated in the two preceding articles afford a ready means of calculating

the smallest allowable width W and clear distance c between webs of compression members when the greatest allowable value of $\frac{l}{r}$ is known. For example, if it is specified that $\frac{l}{r}$ shall not exceed 80, then, for the maximum allowable value of the ratio, $\frac{l}{r} = 80$, and $r = \frac{l}{80}$. For a member whose length in 25 feet, or 300 inches, $r = 300 \div 80 = 3.75$ inches. If the form (f), Fig. 5, is used, r is approximately equal to $\frac{1}{3}W$; and, as r is 3.75 inches, $W = 3 \times 3.75 = 11.25$ inches, and $c = \frac{2}{3} \times 11.25 = 7.5$ inches. In this case, W would probably be made 12 inches or more, and c, 8 inches or more, according to other details.

48. In the forms (j), (m), and (n), r_x is invariably less than r_y, and in the forms (k) and (l), r_y is invariably less than r_x; only the former need be computed if only the least radius of gyration is required.

49. The forms shown in Fig. 5 are frequently modified by the addition of other simple sections, principally plates. The methods of calculation are precisely the same as described in the preceding articles, and should present no further difficulty. It should be remembered that, in finding the radius of gyration of a built-up section, both values should be found if there is the least doubt as to which is the smaller.

50. Location of Center Line.—As already explained, when the stresses in the members of a truss are to be determined, each member is represented by a line. These lines are called the **center lines** of the members For vertical and inclined members, the center line passes through the center of gravity of the section; for chord members, the center line lies close to the center of gravity, but does not pass through it. The distance from the center line of a member to the center of gravity of the cross-section of the member is called the **eccentricity** of the member. The center line of a built-up chord member is located *below the center of gravity in compression members*, and *above the center of*

gravity in tension members, the eccentricity being such that the bending moment on the member due to the eccentricity of the stress shall be equal and opposite to that due to the weight of the member.

The eccentricity e, for chord members, is found by dividing the bending moment M_w of the member due to its own weight by the total stress S in the member; that is,

$$e = \frac{M_w}{S} \qquad (1)$$

If W and l are, respectively, the weight and length of the member, then (see *Strength of Materials*, Part 1),

$$M_w = \frac{W l}{8}$$

This value in formula **1** gives

$$e = \frac{W l}{8 S} \qquad (2)$$

If the weight of the member per foot is denoted by w, then $W = w\, l$, and, therefore,

$$e = \frac{w\, l^2}{8 S} \qquad (3)$$

EXAMPLE.—The gross section of a member whose length is 20 feet is 20 square inches. To find the eccentricity of the center line if the total stress is 200,000 pounds.

SOLUTION —As the gross section of the member is 20 sq. in., its weight per linear foot is $20 \times 3.4 = 68$ lb. (Art. **15**), and, therefore, applying formula **3**,

$$e = \frac{68 \times 20^2}{8 \times 200{,}000} = .017 \text{ ft.} = .204 \text{ in.} \quad \text{Ans.}$$

RIVETING, LATTICE BARS, AND TIE-PLATES

51. Riveting.—Wherever two portions of a built-up member come in contact, a row of rivets is driven, as represented in Fig. 9, in which (*a*) is the top view or plan, (*b*) the side elevation, (*c*) a horizontal section *q q* and bottom view, and (*d*) a vertical section *g g*, of a short piece of an upper chord, and in Fig. 10, which is a short piece of a compression web member, (*a*), (*b*), (*c*), and (*d*), representing the same views as in Fig. 9—that is, a top view, side elevation, longitudinal

section and bottom view and cross-section, respectively. The vertical legs of the flange angles are riveted to the webs with rivets located on the gauge lines *ee*, Fig. 9 (*b*) and Fig. 10 (*b*), of the angles; the cover-plate is riveted to the

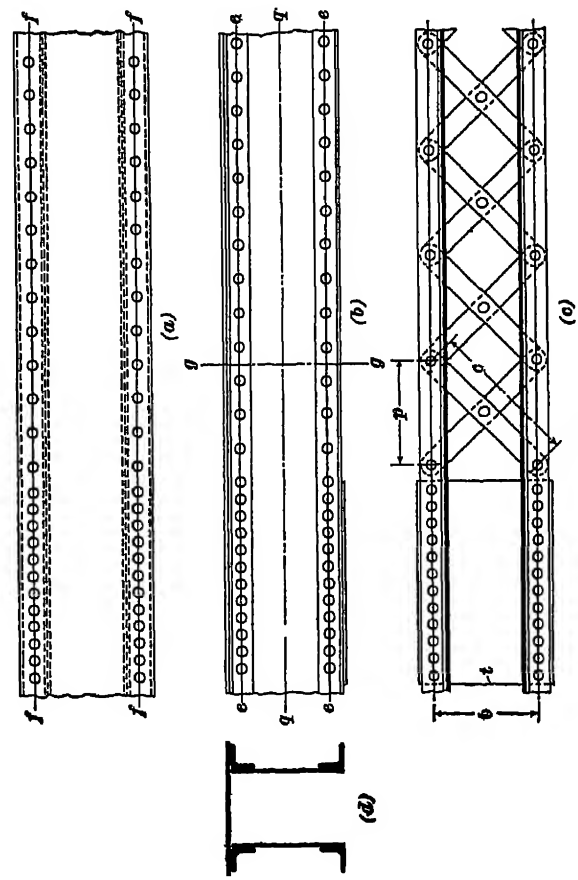


Fig. 9

horizontal legs, frequently called the **outstanding legs**, of the upper flange angles with rivets located on the gauge lines *ff* of the angles, Fig. 9 (*a*) and Fig. 10 (*a*). For a short distance at each end, near where the member connects at the joint, as shown at the left end of Figs. 9 and 10, the

rivets are spaced about 3 inches apart; for the remainder of the distance, they are usually spaced 6 inches apart. The rivets in the horizontal legs of the flange angles are usually located half way between those in the vertical legs, to facili-

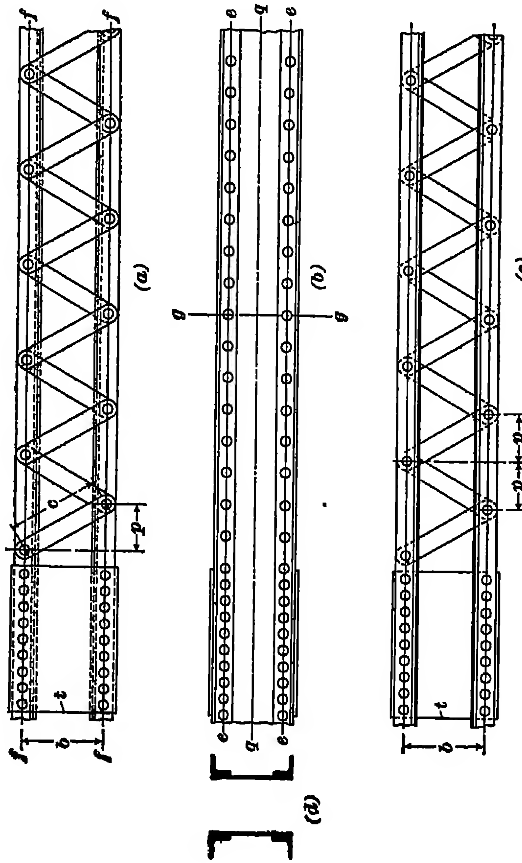


Fig 10

tate driving them; this is called **staggering the rivets** in the two legs.

52. Lattice Bars.—The open sides of built-up members are connected to each other by diagonal bars whose ends are riveted to the outstanding legs of the flanges, as shown

in Fig. 9 (c), and in Fig. 10 (a) and (c). These diagonals are called **lattice bars.** In Fig. 9 (c), the bars cross each other and are riveted at the intersection; this construction is called **double latticing.** The method of latticing shown in Fig. 10 (a) and (c) is called **single latticing.** In double latticing, the flange is divided into a number of equal spaces of such length that the lattice bars will make an angle of about 45° with the axis of the member; in single latticing, the flange is divided into a number of equal spaces of such length that the lattice bar will make an angle of about 60° with the axis of the member. The longitudinal distance, or distance parallel to the axis of the member, between the connections of the ends of a lattice bar is called the **pitch.**

The distance c between the centers of the holes in the ends of a lattice bar can be found by the formula

$$c = \sqrt{p^2 + b^2}$$

in which b = transverse distance between gauge lines of rivets that connect lattice bars to flanges;
p = pitch of latticing.

53. Lattice bars are made from flat bars from 2 to 4 inches in width and from $\frac{1}{4}$ to $\frac{5}{8}$ inch in thickness; the ends are rounded to eliminate sharp protruding corners. It is usually required that the thickness of lattice bars shall be not less than one-fortieth of the distance between the centers of the rivets that connect them to the flanges for single latticing, or one-sixtieth for double latticing. No formula can be given from which to determine when to use single and when double latticing; nor is there any formula for the size of lattice bars. These matters are controlled largely by practice, and depend to a great extent on the width of the member. An approximate rule that agrees fairly well with practice is to use single latticing in all cases where the bars are less than about 15 inches in length, and double latticing in all other cases. The usual locations of the latticing are shown by dotted lines in Fig. 5.

54. Tie-Plates.—For a short distance at each end of a built-up member, near where it is connected at the joints, the

latticing is omitted and the flanges are connected by **tie-plates,** as represented at *t*, Fig 9 (*c*) and Fig. 10 (*a*) and (*c*). These plates are placed as close to the ends of the members as the connections will permit, and are usually made from one to two times as long as they are wide, and from $\frac{1}{4}$ to $\frac{1}{2}$ inch in thickness The plates are connected to the flanges by rivets spaced from 3 to 4 inches apart. The sizes and lengths of tie-plates depend, to a great extent, on practice. They are used in connection with lattice bars to keep the stress evenly distributed between the sides of the member.

GROSS AND NET SECTIONS

55. Definition.—The areas of cross-sections that have been used in the preceding articles are the **gross areas,** commonly called **gross sections.** When a portion of the sectional area of a built-up member is cut away for any purpose, the section is decreased, and the remainder is called the **net section.** In the case of rivet holes, as they are completely filled with rivets, the member offers about as much resistance to compression after the rivets are driven as before the holes were punched, and so the gross area is available in resisting compression, except where there are pinholes and bolt holes; as these are not completely filled, their areas must be deducted. Rivets, bolts, and pins cannot transmit tension from one side of the hole to the other, however, and so only the net section is available in resisting tension.

56. Deduction of Holes.—In finding the net section of a member, it is customary to deduct from the gross section the area of cross-section of the largest number of holes that can be cut by a plane at right angles to the axis of the member. As the holes are $\frac{1}{16}$ inch larger in diameter than the rivets, and as the metal immediately surrounding the holes is somewhat injured by punching, it is common practice to deduct for each hole the area of cross-section of a hole $\frac{1}{8}$ inch larger in diameter than the rivet. Table XXVII gives the area of section to be deducted for each hole for

rivets from $\frac{1}{2}$ to 1 inch in diameter and material $\frac{1}{4}$ inch to $1\frac{1}{2}$ inches in thickness; the areas are found from the formula

$$A = t\left(d + \tfrac{1}{8} \text{ inch}\right)$$

in which A = area, in square inches;
d = diameter of rivet;
t = thickness of metal.

For example, the plane gg, Fig. 10, cuts four rivets—two shown in the figure and the two corresponding on the other side—each of which passes through a web and a flange angle. Then, in order to get the net section, there must be deducted from the gross section the areas of four holes in the webs and four holes in the angles. Suppose that the webs are $\frac{5}{8}$ inch thick, angles $\frac{1}{2}$ inch thick, and rivets $\frac{7}{8}$ inch in diameter. From Table XXVII, it is seen that, for a $\frac{7}{8}$-inch rivet in material $\frac{1}{2}$ inch thick, there must be deducted .50 square inch, and in material $\frac{5}{8}$ inch thick, .625 square inch; then, the total area to be deducted in this case is

$$(4 \times .50) + (4 \times .625) = 4.5 \text{ square inches}$$

If the gross section is 35 square inches, the net section will be $35 - 4.5 = 30.5$ square inches.

57. It is evident also that, if a rivet lies very close to the plane that cuts the rivets just referred to, the section shall be still further decreased. For such cases, the following rule has been found in actual practice to give fairly accurate results for rivets $\frac{7}{8}$ inch in diameter or larger:

Rule.—*To find the net section of a built-up member when the gross section is known, consider the member cut by a plane at right angles to the member in such a position that it will pass through the centers of the largest number of rivets; deduct from the gross section the area of cross-section of one hole for each rivet whose center lies within $\frac{3}{4}$ inch of the plane, and a proportionate part of one hole for each rivet whose center lies within $2\frac{3}{4}$ inches.*

For example, if the center of a rivet is $\frac{3}{4}$ inch from the plane, the entire area for one hole is to be deducted; if $1\frac{3}{4}$ inches, one-half the area for one hole; if $2\frac{1}{4}$ inches, one-quarter the area; and if $2\frac{3}{4}$ inches, no deduction is to be made. For rivets $\frac{3}{4}$ inch in diameter or smaller, the

foregoing limits, $\frac{3}{4}$ inch and $2\frac{3}{4}$ inches, should be changed to $\frac{1}{2}$ inch and 2 inches, respectively.

EXAMPLE —A tension member consisting of one plate 12 in. × $\frac{9}{16}$ in, has its section reduced by the group of $\frac{7}{8}$-inch rivets shown in Fig 11 Find (*a*) the gross section of the plate; (*b*) the net section

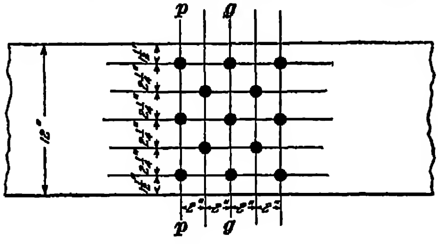


FIG 11

SOLUTION —(*a*) The gross-section of a plate 12 in × $\frac{9}{16}$ in (Table VI) is 6 75 sq in

(*b*) Consider the plate cut by the plane *pp* that passes through the centers of three rivets The area to be deducted for one $\frac{7}{8}$-inch rivet in material $\frac{9}{16}$ inch in thickness is, from Table XXVII, 563 sq in , as there are three rivets cut by the plane, the deduction for them is 3 × 563 = 1 689 sq in In addition, there are two rivets whose centers lie 2 inches from the plane, for each of which, according to the rule, there must be deducted three-eighths of the area for one rivet, the deduction for these holes is, then, 2 × $\frac{3}{8}$ × 563 = 422 sq. in. Then, the total deduction is 1.69 + 42 = 2 11 sq in., and the net section is

$$6\ 75 - 2\ 11 = 4\ 63 \text{ sq in. Ans.}$$

If the plane *gg* is considered instead of *pp*, it can be seen that there are four rivets whose centers are 2 in from the plane In such a case, when the rivets are on different sides of the plane, it is customary to consider only those on one side.

58. Countersunk Rivets.—The holes for countersunk rivets are larger than those for rivets with full head. The last line in Table XXVII gives the amount by which the area of cross-section of a rivet hole is increased for a countersunk head for rivets from $\frac{1}{2}$ to 1 inch in diameter. In finding the net section of a member in which there are countersunk heads, these additional areas should be allowed for.

EXAMPLES FOR PRACTICE

1. From Table XXVI, find the radius of gyration with respect to an axis parallel to the adjacent legs and half way between them for: (*a*) two 4″ × 3″ × $\frac{1}{2}$″ angles with the short legs $\frac{1}{2}$ inch apart and

parallel, (*b*) two 6″ × 4″ × ¾″ angles with the short legs ¾ inch apart and parallel

Ans { (*a*) 2 01 in
(*b*) 3.09 in.

2 The built-up section shown in Fig 12 is composed of one 20″ × ½″ plate and two 15-inch 40-pound channels arranged as shown. Find: (*a*) the distance of the center of gravity from the upper flanges of the channels; (*b*) the moment of inertia with respect to the axis *X′ X*; (*c*) the moment of inertia with respect to the axis *Y′ Y*, (*d*) the radius of gyration with respect to the axis *X′ X*, (*e*) the radius of gyration with respect to *Y′ Y*.

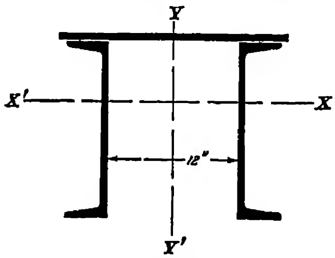


FIG. 12

Ans { (*a*) 5 19 in
(*b*) 1,117 in.⁴
(*c*) 1,433 in.⁴
(*d*) 5.77 in.
(*e*) 6.54 in.

3 The built-up section shown in Fig 13 is composed of two 16″ × ⅝″ plates and four 4″ × 4″ × ⅝″ angles, arranged as shown. Find. (*a*) the moment of inertia with respect to the axis *X′ X*, (*b*) the moment of inertia with respect to the axis *Y′ Y*, (*c*) the radius of gyration with respect to the axis *X′ X*, (*d*) the radius of gyration with respect to *Y′ Y*.

Ans. { (*a*) 1,331 in.⁴
(*b*) 1,411 in ⁴
(*c*) 5 88 in.
(*d*) 6 06 in.

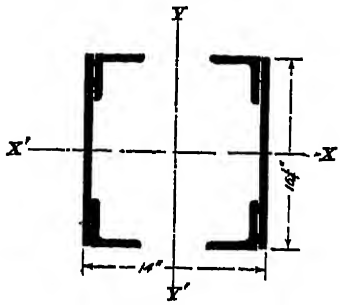


FIG 13

4. A tension member consisting of two 12-inch 30-pound channels has its sectional area reduced by the

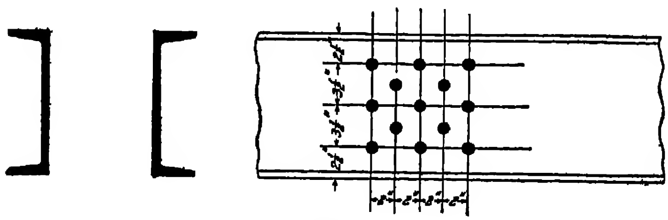


FIG 14

group of ¾-inch rivets shown in Fig 14, in each web. Find the net section of the member.

Ans 14.96 sq. in.

5. A tension member is composed of two plates 15 in $\times \frac{1}{2}$ in. and 8 in. $\times \frac{9}{16}$ in , respectively, and four angles $3\frac{1}{2}$ in. $\times 3\frac{1}{2}$ in. $\times \frac{9}{16}$ in ,

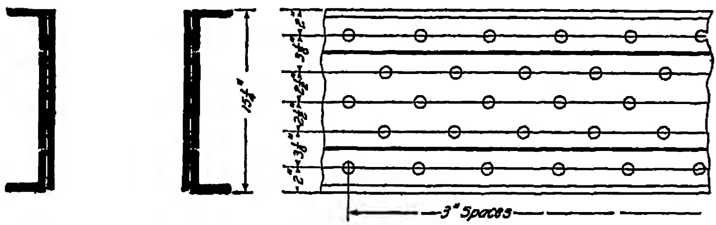


FIG 15

arranged as shown in Fig 15. Find the net section if the rivets in each side are $\frac{7}{8}$ inch in diameter and spaced as shown.

Ans 32.10 sq. in.

PINS, EYEBARS, AND LATERALS

PINS

59. Truss Pins and Nuts.—In pin-connected trusses, a large circular hole is bored at right angles to the truss through each member that connects at a joint, and a pin, somewhat resembling a large bolt, and having a diameter about $\frac{1}{32}$ inch less than that of the hole, is passed through to hold the members in place. The projecting ends of the pin are smaller in diameter than the body of the pin, and a large nut is turned on to each end to keep the members packed together.

The distance between the outside surfaces of the outside members that connect at a joint is called the **grip** of the pin. In order to give the outside members a good bearing, the main body of the pin is made about $\frac{1}{4}$ inch longer at each end than the grip, and the nuts are recessed so that they enclose the projecting ends and bear firmly against the outside members.

Table XXVIII gives the standard dimensions of screw ends and nuts for truss pins varying in diameter from 2 to 8 inches.

60. Lateral Pins.—For the connections of members of lateral and sway systems in pin-connected trusses, a pin

without nuts, often called a **lateral pin,** is sometimes used, although the type of pin described in the preceding article is to be preferred. Lateral pins are turned from rods of slightly larger diameter than the required diameter of the pin, leaving a shoulder at one end; the other end is tapered slightly to facilitate passing the pin through the hole. The tapered end is held in place by a split key called a **cotter pin.** Table XXIX shows the dimensions of heads and cotter pins for the usual sizes of lateral pins.

EYEBARS

61. General Description and Properties.—The principal dimensions of **eyebars,** a special form of tension member used in pin-connected trusses, are given in Table XXX. These bars are first rolled to uniform width and thickness, then the ends are heated and upset to circular forms called **heads,** the thickness remaining the same. Holes are bored in the heads so that they can connect to the pins at the joints. The sizes of heads given in the table were determined from the results of numerous experiments on eyebars, and are those that have proved in actual practice to be sufficient to develop the full strength of the respective bars. In designing tension members for pin-connected trusses, it is, therefore, simply necessary to consider the strength of the body of the bar, as the heads will be strong enough.

The right-hand column of Table XXX gives the additional length U of bar required beyond the center of the pin hole to form one head. The widths given in the first column of the table are standard widths and no others should be used in designing. The thicknesses given in the second column are the minimum thicknesses; any greater thickness can be obtained.

When thicknesses greater than about 2 inches are required, it is customary to use two or more bars side by side. The diameters of pinholes given in the fourth column are the largest holes that can be bored in the heads of the respective bars without decreasing the strength Any smaller hole may be used.

EXAMPLE —An eyebar 6 inches wide and 1 inch thick connects to two pins 6 inches in diameter and 18 feet 6 inches center to center. To find the required length of bar before upsetting the ends to form the heads.

SOLUTION —It will be seen from Table XXX that the first 6-inch bar cannot have a pin larger than $5\frac{1}{4}$ in in diameter in its heads, so it is necessary to consider the second. This allows a pin $6\frac{1}{4}$ in in diameter, and, as the pins in the example are 6 in., this size of head may be used In the right-hand column, the additional length necessary to form one head is given as 2 ft 3 in., as there are two heads, the length of the bar before upsetting is

$$18 \text{ ft } 6 \text{ in } + 2 \times (2 \text{ ft. } 3 \text{ in }) = 23 \text{ ft} \quad \text{Ans.}$$

62. Adjustable Eyebars.—When there are two diagonals in the same panel, that is, a main diagonal and a counter, it is difficult to manufacture them so that all the holes in their ends will exactly fit the pins In such cases, it is customary to make the counter adjustable; that is, to manufacture it in such a way that its length can be slightly increased or decreased, if desired. Table XXXI gives the principal dimensions of **adjustable eyebars**; they are used in pairs, one end of each bar having a flat circular head and pinhole, as given in Table XXX, and the other an upset screw end, as given in Table XXXI. The additional length of bar necessary to form an upset screw end is given in Table XXXI; that necessary to form a flat circular head may be taken from Table XXX. Owing to the method of manufacture, it is impossible to get an adjustable eyebar less than about 7 feet in length; for this reason, in designing, it is well to make one end of an adjustable member about 7 feet long.

63. Turnbuckles.—The screw ends of the two adjustable eyebars that form a counter are brought to within 3 inches

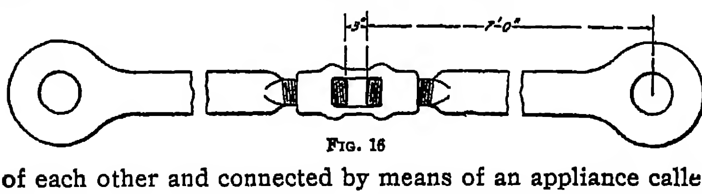


FIG. 16

of each other and connected by means of an appliance called a **turnbuckle** (see Fig. 16). The bars are screwed into the turnbuckle until the member has the desired length. The

threads on the ends of the two bars run in opposite directions, so that, by twisting the turnbuckle, the length of the member may be increased or decreased as desired. Table XXXII gives the principal dimensions of the turnbuckles that are most used in bridge work at the present time. They are called **open turnbuckles**, to distinguish them from less desirable forms, called **closed turnbuckles**, that resemble long nuts. Turnbuckles are made of wrought iron, and are manufactured of such dimensions as will give them greater strength than the bars for which they are designed.

EXAMPLE.—A counter connects to two $3\frac{3}{4}$-inch pins, 22 feet center to center, and is composed of a pair of adjustable eyebars 4 inches wide and $\frac{3}{4}$ inch thick. What is the length of each part before upsetting?

SOLUTION.—The short end, after upsetting, will be 7 ft. long, and the other end, 22 ft. − 7 ft. 3 in. = 14 ft. 9 in. long. Table XXXI gives the additional length necessary to form a screw end as 1 ft. $\frac{3}{4}$ in.; Table XXX gives the additional length necessary to form a flat circular head as 1 ft. $5\frac{1}{2}$ in. Then, the original length of the short end is

7 ft. + 1 ft. $\frac{3}{4}$ in. + 1 ft. $5\frac{1}{2}$ in. = 9 ft. $6\frac{1}{4}$ in.

and that of the long end is

14 ft. 9 in. + 1 ft. $\frac{3}{4}$ in. + 1 ft. $5\frac{1}{2}$ in. = 17 ft. $3\frac{1}{4}$ in. Ans.

LOOP-WELDED AND LATERAL RODS

64. Loop-Welded Rods.—Fig. 17 shows another form of member sometimes used for counters and lateral rods, although it is being superseded by the adjustable eyebar. It is called a **loop-welded rod**, and is made from a straight

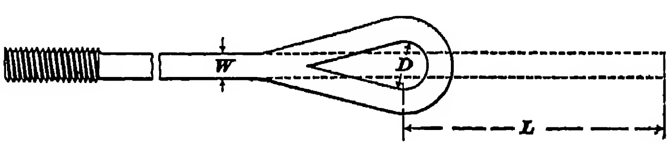


FIG. 17

square rod; one end is upset for a screw end, and the other made in the form of a loop by bending around a circular pin and welding the free end to the main body of the bar. On account of the difficulty and uncertainty in welding steel

these rods are usually made of wrought iron They are used in pairs connected by a turnbuckle in the same way as adjustable eyebars. The additional length of rod required to form an upset screw end can be found from Table III; that required to form a loop end is given approximately by the formula

$$L = 3.75\,(W + D)$$

in which W = width of bar, in inches;
D = diameter of pin, in inches;
L = additional length beyond center of pin hole, in inches.

EXAMPLE.—A loop-welded counter consisting of a 2-inch square rod connects on two 4-inch pins 21 feet apart To find the required lengths of straight rod to form the member

SOLUTION —One end may be made 7 ft long, and the other 21 ft − 7 ft 3 in , or 13 ft 9 in long Table III gives the additional length to form an upset screw end as $4\frac{3}{4}$ in By the formula, the additional length to form a loop end is $3\ 75 \times (4 + 2) = 22\ 50$ in , or 1 ft $10\frac{1}{2}$ in. Then, the length of the short piece must be

$$7 \text{ ft.} + 4\tfrac{3}{4} \text{ in} + 1 \text{ ft } 10\tfrac{1}{2} \text{ in.} = 9 \text{ ft } 3\tfrac{1}{4} \text{ in.},$$

and that of the long piece,

$$13 \text{ ft. } 9 \text{ in} + 4\tfrac{3}{4} \text{ in.} + 1 \text{ ft } 10\tfrac{1}{2} \text{ in} = 16 \text{ ft. } \tfrac{1}{4} \text{ in.} \quad \text{Ans.}$$

65. Lateral Rods.—Table XXXIII gives the principal dimensions of flat circular heads, similar to eyebar heads, for square and round rods, as sometimes used for the connection of members of lateral and sway systems. When so used, the rods are sometimes made in one piece, with a head at each end, and sometimes in two pieces, connected by a turnbuckle. In the latter case, there should be 3 inches between the ends of the pieces for adjustment. As in the case of adjustable eyebars, it is well to make one end shorter than the other, but the short end should be not less than 7 feet in length

66. Clevises.—Table XXXIV gives the principal dimensions of clevises used for connecting lateral rods to pins to which other lateral rods connect. Instead of forming a head on the end of the rod, an upset screw end is formed, which is screwed into the clevis.

67. Types of Lateral Rods.—Various combinations are used for lateral rods, such as a single rod with flat circular heads, as in Fig. 18 (*a*); a pair of rods with flat circular heads and connected by a turnbuckle, as in Fig. 18 (*b*); a single rod with clevises at both ends, as in Fig. 18 (*c*), and

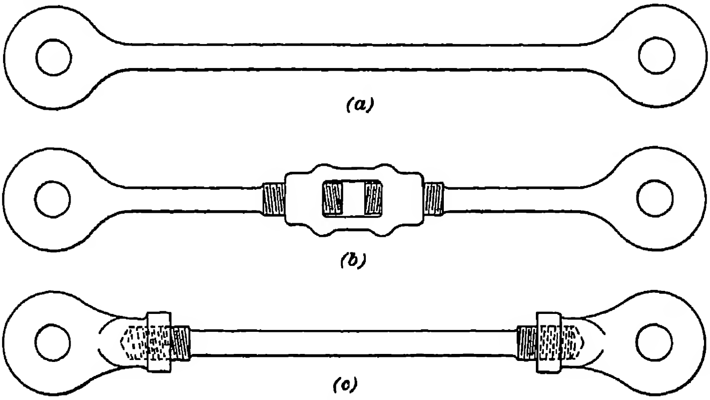


FIG 18

a pair of rods with clevises at one or both ends and connected by a turnbuckle. The type used depends on other details. Lateral rods are being superseded by single angles or built-up members. It is found in practice that the latter are stiffer than lateral rods and therefore more desirable.

EXAMPLES FOR PRACTICE

1. An eyebar 5 inches wide and $\frac{7}{8}$ inch thick connects to two pins 5 inches in diameter and 20 feet center to center Find the required length of bar before upsetting the ends to form the heads. Ans. 24 ft.

2 A counter connects to two $3\frac{1}{2}$-inch pins 27 feet center to center, and is composed of one pair of $3'' \times \frac{7}{8}''$ eyebars connected by a turnbuckle Find the length of each part before upsetting.

Ans. $\begin{cases} \text{9 ft 5 in.} \\ \text{22 ft. 2 in.} \end{cases}$

3 A loop-welded counter consisting of a $2\frac{1}{4}$-inch square rod connects to two 3-inch pins 30 feet apart. Find the required length of each part before the loops are formed.

Ans. $\begin{cases} \text{9 ft. } \frac{13}{16} \text{ in} \\ \text{24 ft. } 9\frac{13}{16} \text{ in.} \end{cases}$

WORKING STRESSES

68. Introductory.—In *Strength of Materials*, Part 1, the nature and use of the ultimate strength and factor of safety were explained. At one time, it was not an uncommon practice to use the ultimate strength and factor of safety in the design of bridge members. At present, however, it is common practice to make use of certain allowable intensities of stress, called **working stresses**, which vary according to the different kinds of stress and the different conditions of loading The stresses are based on the results of experiments, and are so chosen as to allow a sufficient factor of safety without further reference to it. The allowable working stresses are usually stated in bridge specifications, which require that all parts shall be so designed that the maximum stresses will not cause the actual intensities of stress to exceed the working stresses

69. Tension.—The intensity of tensile stress in any member is found by dividing the total stress by the net area of cross-section of the member. For example, if the total stress in a member is 100,000 pounds tension, and the net section is 10 square inches, the intensity of stress is 100,000 ÷ 10 = 10,000 pounds per square inch. In designing, the required net section of a tension member is found by dividing the total stress by the allowable working stress. For example, if the total stress in a member is 288,000 pounds tension, and the working stress for tension is 16,000 pounds per square inch, the required net section of the member is 288,000 – 16,000 = 18 square inches.

70. Compression.—The working stresses allowed in compression for short members are usually the same as those in tension. For members whose length is greater than eight times the least diameter (that is, the least transverse dimension), the allowable working stress is reduced

to provide for the tendency of the column to buckle or bend sidewise. In *Strength of Materials*, Part 2, it was explained that a column with fixed ends is stronger than one with the ends hinged or not fixed The compression members of pin-connected trusses, except the top chord when made continuous, are hinged or free to turn around the pins in their ends The compression members of riveted trusses are fixed in direction at the ends by being riveted to the gussets or connection plates at the joints, and are, therefore, theoretically stronger than pin-connected columns. When riveted trusses deflect, however, the gussets or connection plates tend to twist, and this twisting produces bending in the members that connect to the gussets, giving rise to what are known as **secondary stresses**, whose magnitudes cannot be computed. On this account, the advantage of riveted columns over pin-connected columns in bridge work is ignored, and the same reduction formula is used to find the allowable working stress in both forms.

71. Various formulas for compression are in use at the present time, all of them being based on the ratio of the unsupported length of the column to the least radius of gyration of its cross-section. The formula now most frequently used in practice has the following general form:

$$s_c = \frac{s}{1 + \frac{l^2}{c\,r^2}}$$

in which s_c = allowable working stress for column;
l = unsupported length of column;
r = least radius of gyration of cross-section;
c = a constant determined by experiment.

Different specifications give different values for s and c.

In *Bridge Specifications*, s is taken as 16,000 pounds per square inch, and c, as 18,000. The formula then becomes

$$s_c = \frac{16{,}000}{1 + \frac{l^2}{18{,}000\,r^2}}$$

The values of l and r should be given in the same units; that is, both should be in feet, or both in inches. For

example, if l is 12 feet, or $12 \times 12 = 144$ inches, and r is 3 inches,

$$s_c = \frac{16{,}000}{1 + \frac{144 \times 144}{18{,}000 \times 3 \times 3}} = \frac{16{,}000}{1.128} = 14{,}180 \text{ lb. per sq. in}$$

Table XXXV gives the values of s_c for various values of $\frac{l}{r}$.

72. In designing compression members, it is customary to decide on the general form of the cross-section, and then determine the approximate value of the radius of gyration and dimensions of the section, as explained in Art. **47**; the length of the member is generally known, and the approximate value of $\frac{l}{r}$ can be computed. The allowable working stress corresponding to this value of $\frac{l}{r}$ is then found from Table XXXV, and the required area of cross-section found by dividing the total compression by the allowable working stress. With this area as a basis, the cross-section is determined, its radius of gyration and the actual value of $\frac{l}{r}$ are computed, and the working stress corresponding to the corrected value of $\frac{l}{r}$ is found from Table XXXV. The area of cross-section is then revised, if necessary, to make it correspond with the new working stress.

EXAMPLE —The total compressive stress in a member 20 feet long is 281,000 pounds, and the approximate value of the radius of gyration is 4 8 inches What is the trial value of the area of cross-section of the member that would be used in designing?

SOLUTION —The length of the member is 20 ft , or $20 \times 12 = 240$ in The value of $\frac{l}{r}$ is $\frac{240}{4.8} = 50$. Consulting Table XXXV, the working stress corresponding to a value of 50 for $\frac{l}{r}$ is found to be 14,050 lb per sq. in. Then, since the total compressive stress is 281,000 lb , the required area of cross-section is

$$281{,}000 \div 14{,}050 = 20 \text{ sq in.} \quad \text{Ans}$$

73. Shearing.—The intensity of shearing stress on any member subjected to a shearing force at right angles to its

length is usually found by dividing the stress by the area of cross-section of the member. In designing, the shearing stress and the allowable working stress are known, and the required area of cross-section is found by dividing the shearing stress by the working stress.

74. In wide thin plates subject to a shearing stress in the plane of the plate, there is a tendency to buckle, or bend outwards, and when the intensity of stress is sufficient, this is resisted by stiffening angles *a*, *a*, *a*, Fig 19, riveted to the plate and parallel to the direction of the shearing stress.

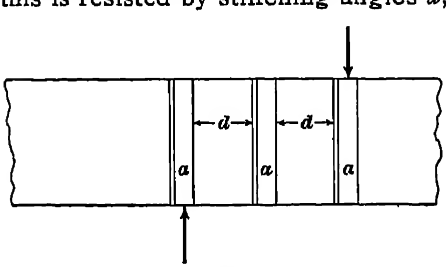


Fig 19

The allowable working stress for shearing in wide plates is found by means of a formula similar to the formula used in finding the working stress for compression. Different engineers make use of different formulas, the one most used in bridge work at the present time being

$$s_s = \frac{12{,}000}{1 + \dfrac{d^2}{3{,}000\,t^2}}$$

in which s_s = working stress;

t = thickness of plate,

d = width of plate, or, if plate is stiffened by angles closer together than width of plate, the clear distance between the stiffener angles, as shown in Fig 19.

For example, if a plate 24 inches wide and ½ inch thick, with no stiffeners or with stiffeners farther apart than 24 inches, is subject to a shearing stress, the allowable working stress for shear is

$$s_s = \frac{12{,}000}{1 + \dfrac{24 \times 24}{3{,}000 \times .5 \times .5}} = 6{,}790 \text{ lb. per sq. in.}$$

If, however, the stiffeners are closer together than the width of the plate, say 12 inches apart, the allowable working stress for shear is

$$\frac{12{,}000}{1+\dfrac{12\times 12}{3{,}000\times .5\times 5}} = 10{,}070 \text{ lb. per sq. in.}$$

75. Table XXXVI gives curves that show the allowable working stresses in plates from $\frac{1}{4}$ to 1 inch in thickness, and from 0 to 100 inches in width, calculated from the formula in Art. **74.** To find by means of the curve the allowable working stress in the first case considered above, we glance at the left-hand side and find 24, corresponding to an unsupported width of 24 inches; we then glance horizontally across to the curve for a $\frac{1}{2}$-inch plate, marked $\frac{1}{2}$, and then vertically downwards to the bottom line, on which we find 6,800, which is the allowable working stress, in pounds per square inch.

In the case of plate girders, the flange angles are riveted to the edge of the web, and the unsupported distance is taken equal to the clear distance between the vertical legs of the flange angles, or between stiffeners, whichever is the smaller.

76. Bending.—The allowable working stress in bending on rolled sections is usually the same as that allowed in tension When the unsupported length of the compression portion is greater than a certain number of times its width, usually twenty times, it is customary to decrease the allowable working stress in compression by means of some reduction formula. A formula for this purpose will be given elsewhere.

BRIDGE MEMBERS AND DETAILS

(PART 2)

DETAILS

RIVETED JOINTS

RIVET VALUES

NOTE.—The tables referred to in this Section are found in *Bridge Tables*.

1. Introductory.—Rivets are used for two principal purposes; namely, to hold the several parts of a built-up member together so as to make them act as one piece, and to transmit stress from one member or part of a member to another. In the first case, the stresses on the rivets are not calculated; the rivets are spaced according to established practical rules that will be given in *Bridge Specifications* In the second case, the stresses on the rivets and the number of rivets required to transmit given stresses must be computed in order that the allowable stress on a rivet may not be exceeded.

It has been found by experiment that, when the spaces between centers of rivets are less than three times the diameter of the rivet, and the rivets are not nearer the edge of a plate than one and one-half times the diameter, the only conditions of strength that must be considered are the resistance offered by the rivet to shearing on one or more

sections between the members connected, and the resistance to crushing offered by the plates bearing on the rivets.

2. Friction of Riveted Joints.—As explained in *Bridge Members and Details*, Part 1, rivets are driven hot, and in cooling contract and hold firmly together the parts through which they pass. This causes a certain amount of friction between the parts, which helps to transmit stress from one member or part of a member to another. It is impossible to determine with any certainty how great this friction is, and it is customary to ignore it in the design of riveted joints, the shearing and bearing resistances alone being considered.

3. Shearing Value.—The maximum shearing stress that is allowable on a rivet is called the **shearing value** of the rivet, or the **value** of the rivet, in shear. If a rivet connects two plates, as represented in Fig. 1, the stress in one plate is transmitted to the other by means of the rivet, and the area subjected to shear is the area of cross-section of the rivet. As there is but one section *c* of the rivet subjected to shear, the rivet is said to be in **single shear.** When a rivet connects two members, as represented in Fig. 2, the rivet is subjected to shear at two sections *d* and *e*, and is said to be in **double shear.** In calculating the area of cross-section of a rivet, the nominal diameter is used.

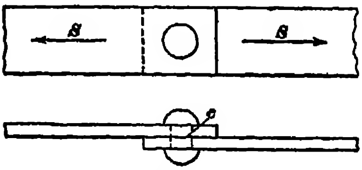


FIG. 1

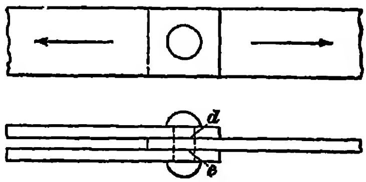


FIG 2

The shearing value of a rivet is found as follows:

Rule.—*To find the value of a rivet in single shear, multiply the area of cross-section of the rivet by the allowable intensity of stress in shear, to find the value of a rivet in double shear, multiply twice the area of cross-section of the rivet by the allowable intensity of stress in shear.*

Example —What is the value of a $\frac{7}{8}$-inch rivet (*a*) in single shear, and (*b*) in double shear, if the working stress is 11,000 pounds per square inch?

Solution —(*a*) Consulting Table I, the area of cross-section of a $\frac{7}{8}$-inch round rod is found to be 601 sq in. Since the working stress is 11,000 lb per sq in , the value in single shear is

$$601 \times 11{,}000 = 6{,}610 \text{ lb} \quad \text{Ans}$$

(*b*) The value in double shear is

$$2 \times .601 \times 11{,}000 = 13{,}220 \text{ lb.} \quad \text{Ans}$$

4. Bearing Value.—The resistance to crushing offered by the members that bear on a rivet depends on the thickness of the plates that transmit the stress to and from the rivet, and on the diameter of the rivet. When a rivet bears on a plate, it is in contact with the inside of the rivet hole, as at *a b c*, Fig. 3, the intensity of bearing or pressure being greater at *b* than at *a* and *c*, and it is difficult to determine the exact area of bearing. In practice, it is customary to assume that the area of bearing is equal to the product of the nominal diameter of the rivet and the thickness of the plate The working strength of the plate is commonly called the **bearing value** of the plate, and is equal to the product of the diameter of the rivet, thickness of the plate, and allowable intensity of stress.

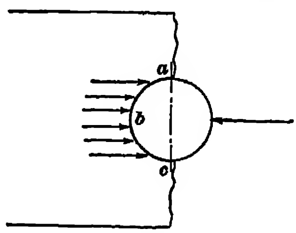


Fig 3

The bearing value of a plate on a rivet is frequently spoken of as the bearing value *of the rivet* on the plate, it being understood that the bearing value *of the plate* is meant.

Example.—If the working stress in bearing is 22,000 pounds per square inch, what is the bearing value of a plate $\frac{1}{2}$ inch thick on a rivet $\frac{7}{8}$ inch in diameter?

Solution —The bearing value is equal to the continued product of the diameter, $\frac{7}{8}$ in., the thickness of the plate, $\frac{1}{2}$ in., and the working stress, 22,000 lb per sq in ; that is, the bearing value is

$$\tfrac{7}{8} \times \tfrac{1}{2} \times 22{,}000 = 9{,}625 \text{ lb} \quad \text{Ans.}$$

5. Tables of Rivet Values.—Tables XXXVII to XL give the values of rivets from $\frac{1}{2}$ to 1 inch in diameter, in

single and double shear, for working stresses of from 6,000 to 11,000 pounds per square inch in shear, and the bearing values of plates $\frac{1}{4}$ to $\frac{11}{16}$ inch in thickness on rivets $\frac{1}{2}$ to 1 inch in diameter, for working stresses from 12,000 to 22,000 pounds per square inch in bearing. For example, if it is desired to know the value in double shear of a rivet $\frac{3}{4}$ inch in diameter for an allowable stress in shear of 9,000 pounds per square inch, Table XXXIX is consulted. In the fourth column from the left, headed Shear Values at 9,000 Pounds per Square Inch, Double Shear, is found, opposite $\frac{3}{4}$ in the first column, the value 7,950 pounds, which is the required value. In like manner, if it is desired to know the bearing value of a $\frac{9}{16}$-inch plate on a $\frac{7}{8}$-inch rivet, for an allowable stress in bearing of 15,000 pounds per square inch, Table XXXVIII is consulted. In the column headed $\frac{9}{16}$, and opposite $\frac{7}{8}$ in the first column, the required value, 7,380 pounds, is found.

In Tables XXXVII to XL, the bearing values below and to the left of the dotted lines are less than the values in single shear; those below and to the left of the heavy full lines are less than the values in double shear; and those above and to the right of the heavy full lines are greater than the values in double shear.

6. Critical Value of a Rivet.—Each rivet may be said to have three values; namely, its shearing value, and the bearing values of the members from which and to which the stress is transmitted. For example, in Fig. 4, the stress is transmitted from the two angles *a*, *a* to the plate *b*. The rivets bear on the two angles, are in double shear, and bear on the plate. If the rivets are $\frac{7}{8}$ inch in diameter, the angles $\frac{3}{8}$ inch thick, the plate $\frac{1}{2}$ inch thick, and the working stresses in shear and bearing are 9,000 and 18,000 pounds per square inch, respectively, Table XXXIX shows that the value in double shear is 10,820 pounds; in bearing on the $\frac{1}{2}$-inch plate, 7,880 pounds; and in bearing on two $\frac{3}{8}$-inch angles ($\frac{3}{4}$ inch total thickness), 11,810 pounds. The smallest of these three values—in this case the bearing on the $\frac{1}{2}$-inch plate—is called the **critical value** of the rivet, and sometimes simply *the* value of the rivet.

7. In making a drawing of a riveted member, it is customary to show all the rivets or to indicate their location in some way. When several views of the member are drawn, the rivets are usually represented by the conventional signs explained in *Bridge Members and Details*, Part 1. When only one view is shown, and frequently also when more views than one are shown, the rivets are represented in other ways. For example, in Figs. 1, 2, and 4, the rivets are represented

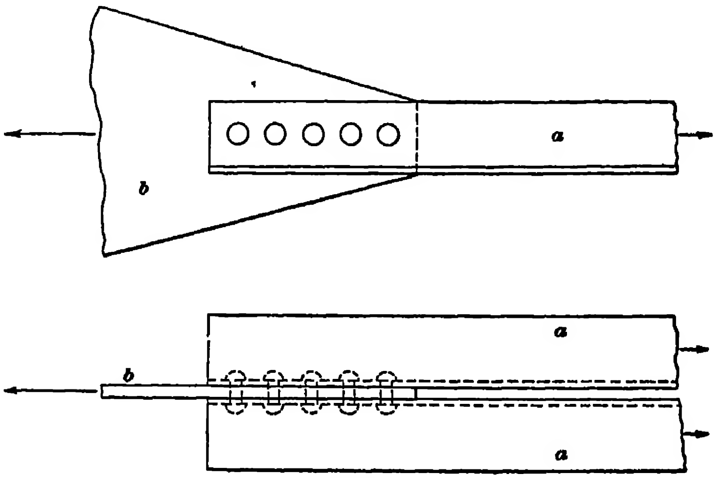


FIG 4

in the upper part of each figure by circles; and in the lower portion they are represented in elevation. In Fig. 5, they are shown in but one view. In Fig. 6, the rivets are shown only in (*a*), but in that view some are shown in plan and some in side elevation. In Fig. 7, the conventional signs are not shown, but the location of each rivet is represented by a small cross; this method is frequently used when there are many rivets to be drawn.

8. **Required Number of Rivets.**—The number of rivets required to connect two members is found by dividing the stress that is to be transmitted, by the critical value of one rivet. For example, if the stress that is

transmitted from the angles to the plate in Fig. 4 is 38,000 pounds, and other conditions are as given in Art. **6**, then, since the critical value of a rivet has been found to be 7,880 pounds, the required number of rivets is 38,000 ÷ 7,880 = 4.82 The next whole number, 5 in this case, is always provided.

EXAMPLE.—A 10-inch 20-pound channel, in which the stress is 82,500 pounds, is connected to a $\frac{7}{16}$-inch plate by $\frac{7}{8}$-inch rivets, as represented in Fig. 5. If the working stresses on the rivet are

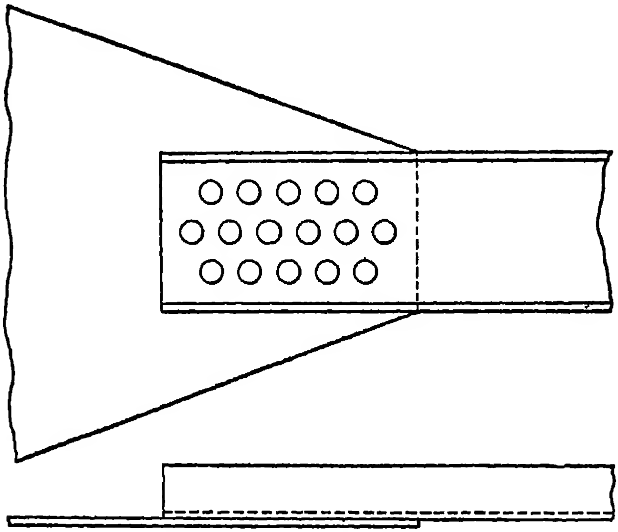


FIG 5

9,000 pounds per square inch in shearing and 18,000 pounds per square inch in bearing, how many rivets are required to transmit the stress from the channel to the plate?

SOLUTION —Consulting Table XIII, the thickness of the web of a 10-in 20-lb channel is found to be $\frac{3}{8}$-in. Consulting Table XXXIX, the value of a $\frac{7}{8}$-in rivet in bearing on a $\frac{7}{16}$-in plate is found to be 6,890 lb ; in single shear, 5,410 lb ; and in bearing on the $\frac{3}{8}$-in. web of the channel, 5,910 lb As the value 5,410 lb of a $\frac{7}{8}$-in rivet in single shear is the smallest, it is the critical value of the rivet Then, since the total stress is 82,500 lb , the required number of rivets is

82,500 ÷ 5,410 = 15.25, or, say, 16 Ans.

SPLICES IN BUILT-UP MEMBERS

9. Length of Members.—Bridge members composed of built-up shapes are occasionally made as long as 80 feet. These members are exceedingly difficult to handle, both on account of their weight and on account of their size. Their weight, too, is likely to cause excessive stresses while they are being handled In general, it is advisable to manufacture long members, such as chords of trusses, in several sections, each being from 30 to 60 feet in length. The ends of the several sections are planed so that they will bear against one another, and additional plates and angles, called **splice plates** and **splice angles**, are riveted to the ends of the members where they come together.

10. Forms of Splices.—Fig. 6 shows the elevation (*a*) and cross-section (*b*) of a spliced member composed of two

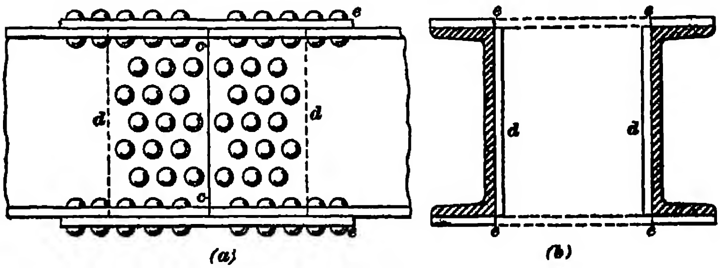


FIG. 6

channels. The ends of the two sections are brought together and placed in contact at *c*, *c*. The plates *d*, *d* are riveted to the inside of the channels, and the plates *e*, *e* to the flanges of the channels. Both *d* and *e* are splice plates. In some cases, the plates *e* are made the full width of the member, as shown by dotted lines. The area of cross-section of the splice plates must in every case, both for tension and for compression members, equal the area of cross-section of the member. Some engineers depend on the bearing area at *c*, *c* between the ends of the two portions to transmit compression, and simply place enough splice plates and rivets at

the joint to hold the member in line The best practice at the present time, however, is to make the area of the splice plates equal to the area of the member, and to place enough rivets in each splice plate to transmit the amount of stress that is transmitted by the plate. Further details regarding splices will be given in subsequent Sections.

JOINTS IN RIVETED TRUSSES

11. Typical Joint.—Fig. 7 represents the top view (*a*), side view (*b*), and end view (*c*) of a typical top chord joint in a riveted truss. In parallel-chord trusses, the top chord *de*

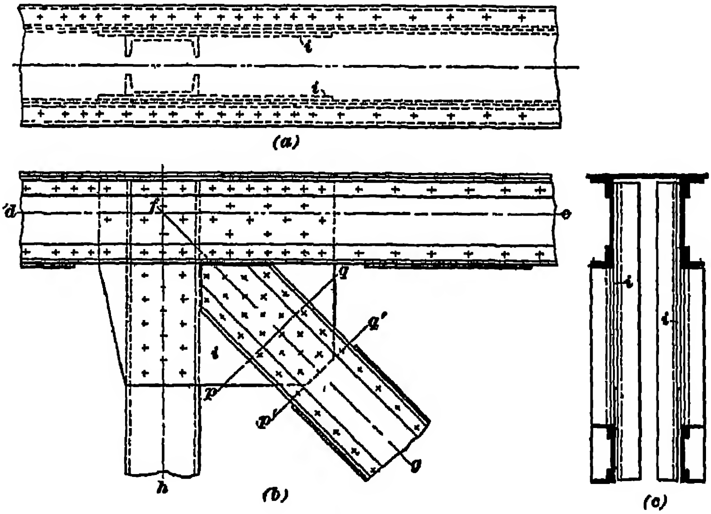


FIG. 7

is made continuous; that is, the angles and plates composing it are not joined at the panel points, unless it is desired to splice it at such places. The line *dfe* near the center of gravity of the chord is called the center line of the member, and the lines *fg* and *fh*, through the centers of gravity of the web members, intersect the line *dfe* and each other at the panel point *f*. The plates *i*, called **gussets**, or **web connection plates**, are riveted to the insides of the chord

member, and the web members are riveted to the inside or outside of these plates; in Fig. 7, fg is shown riveted to the outside and fh to the inside. The location of the web members with respect to the gusset plates depends on other details. Other joints will be described in subsequent Sections.

12. Rivets in Connection Plates.—It is customary to assume that one-half the stress in each web member is transmitted to each gusset. The number of rivets required to connect each side of a web member to a gusset is found by dividing one-half the stress in the web member by the value of one rivet. The stress transmitted to the chord by the connection plate is equal to the algebraic sum of the X components of the stresses in the web members that connect at the joint. It is seldom necessary to actually compute the number of rivets required to connect the gussets to the chords. If the gussets are made large enough to contain sufficient rivets to transmit the stresses to and from the web members, and are riveted to the chords sufficiently to hold them firmly together, say with a rivet pitch of 3 inches, there will invariably be enough rivets.

13. Size of Gussets.—It is impossible to calculate accurately the stresses in the gussets, and, therefore, to determine their required dimensions with exactness. It is well to see, however, that the area of cross-section on such a section as pq, Fig. 7, at right angles to the diagonal, is sufficient to resist the stress transmitted to the gusset at that point. The amount of stress transmitted to the gusset at such a section can be found by multiplying the value of one connecting rivet by the number of rivets in the gusset between the section considered and the edge $p'q'$ of the plate. It is always advisable to have an excess of material in the gusset at the section considered. In practice, gussets should never be made less than $\frac{7}{16}$ inch thick for highway-bridge trusses nor less than $\frac{1}{2}$ inch for railroad-bridge trusses.

14. Clearance.—When a member is riveted to the inside of the gussets, it is customary to make it $\frac{1}{8}$ inch

narrower than the clear distance between the gussets The object of this is to make it easier to push the member between the gussets when the parts are being put together.

PIN JOINTS

15. Typical Pin Joint.—Fig. 8 represents the side elevation (*a*), end view (*b*), and top view (*c*) of a typical bottom chord joint in a pin-connected truss. The bottom

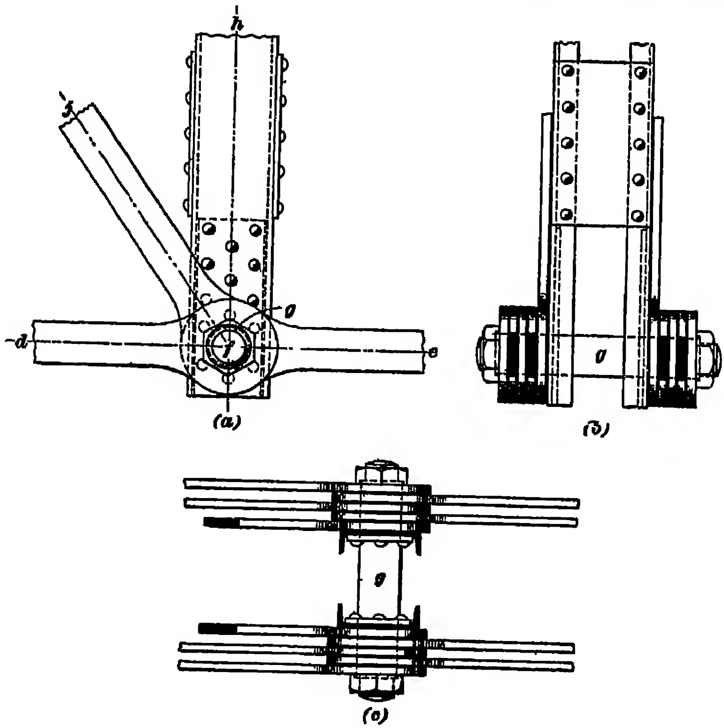


FIG 8

chord *dfe* and diagonal *fj* are made of eyebars; the vertical *fh* is usually composed of two channels. The pin *g*, which passes through the ends of the members, holds them in position and transmits the stresses from one member to

another. The different members that connect to a pin are placed as close together as possible. It is customary to locate the vertical member nearest the center of the pin, then the diagonals, and then the eyebars in the bottom chord, alternating them as shown. The required size of the pin depends on the stresses to be transmitted and on the relative location of the members. Two conditions are considered in the design of a pin; namely, the intensity of bearing on the members, and the bending moment on the pin.

16. Intensity and Required Thickness of Bearing. The area of bearing of a member on a pin is found, in the same way as the bearing area of a rivet, by multiplying the diameter d of the pin by the thickness t or width of the bearing. The intensity of pressure s_b is then found by dividing the total stress S in the member by the bearing area. Eyebars are so designed by the manufacturers that the intensity of bearing need not be considered, if the conditions given in Table XXX are adhered to. In built-up members, however, it is always necessary to compute the intensity of bearing by means of the formula

$$s_b = \frac{S}{t\,d}$$

When the diameter of the pin and the working stress in bearing are known, the required thickness of bearing can be found by the formula

$$t = \frac{S}{s_b\,d}$$

Table XLI gives, in columns 3, 4, and 5, the values of $s_b d$ for all the ordinary sizes of pins and for customary working stresses. In case a member has not the required thickness, the portion in contact with the pin is reinforced by pin plates.

17. Pin Plates.—Pin plates are placed on the members as represented in Fig. 9, in which c, c are the pin plates. It is customary to assume that the stress in a member is evenly distributed over the thickness or width of bearing; so the amount transmitted by the pin plates can be found by multiplying the total stress by the ratio of the thickness of the

pin plates to the total thickness of bearing. The pin plates must be connected to the member by sufficient rivets to transmit this portion of the stress to the member.

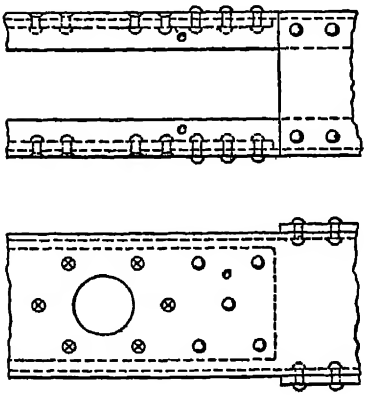


FIG 9

EXAMPLE —The total stress transmitted by a member is 220,000 pounds The thickness of the member in bearing on a 5-inch pin is $\frac{3}{4}$ inch, and the allowable intensity of bearing is 22,000 pounds per square inch (*a*) What is the required thickness of bearing? (*b*) What is the required thickness of pin plates? (*c*) What is the amount of stress transmitted by the pin plates? (*d*) If the rivets connecting the pin plates to the member have a value of 4,860 pounds, how many rivets are required?

SOLUTION.—(*a*) The required thickness of bearing is found by the formula

$$t = \frac{S}{s_b\, d}$$

In the present case, S = 220,000 lb Consulting Table XLI, it is seen that $s_b\, d$ = 110,000 lb. Then,

$$t = 220,000 \div 110,000 = 2 \text{ in. Ans.}$$

(*b*) Since the required thickness is 2 in , and the thickness of the member is $\frac{3}{4}$ in., the required thickness of pin plates is

$$2 - \tfrac{3}{4} = 1\tfrac{1}{4} \text{ in Ans}$$

(*c*) Since the required thickness of pin plates is $1\frac{1}{4}$ in., and the total thickness is 2 in., the amount of stress transmitted by the pin plates is

$$\frac{1\ 25}{2} \times 220{,}000 = 137{,}500 \text{ lb Ans}$$

(*d*) Since the amount of stress transmitted by the pin plates is 137,500 lb , and the value of one rivet is 4,860 lb., the number of rivets required to connect the pin plates to the member is

$$137{,}500 \div 4{,}860 = 28.3, \text{ or, say, } 29. \text{ Ans.}$$

18. Bending Moment on the Pin.—The required diameter d of the pin is found from the maximum bending moment M on the pin, by means of the formula

$$d = \sqrt[3]{\frac{32\, M}{\pi\, s_f}} \qquad (1)$$

in which s_f is the working stress in bending.

As this formula is not convenient for rapid work, it is customary in practice to use a table that gives values of M corresponding to the usual working stresses and diameters of pins Table XLI gives in columns 6 to 10 the values of the allowable bending moments on different pins for several working stresses. Those bending moments have been computed by means of the formula

$$M = \frac{\pi s_f d^3}{32} \qquad (2)$$

For example, if it is known that the bending moment on a pin is 270,000 inch-pounds, and the working stress in bending is 22,000 pounds per square inch, it can be seen by a glance at column 9 that a pin 5 inches in diameter is required. The method of calculating the maximum bending moments on the pins will be given in a subsequent Section.

EXAMPLES FOR PRACTICE

1 If the working stress in shear is 6,000 pounds per square inch, what is the value of a $\frac{3}{4}$-inch rivet (*a*) in single shear? (*b*) in double shear?

Ans $\begin{cases} (a) \ 2{,}650 \text{ lb.} \\ (b) \ 5{,}300 \text{ lb} \end{cases}$

2 If the working stress in bearing is 18,000 pounds per square inch, what is the bearing value of a $\frac{5}{8}$-inch rivet on a plate (*a*) $\frac{5}{16}$ inch in thickness? (*b*) $\frac{5}{8}$ inch in thickness?

Ans $\begin{cases} (a) \ 3{,}520 \text{ lb.} \\ (b) \ 7{,}030 \text{ lb.} \end{cases}$

3 A 9-inch 15-pound channel in which the stress is 39,800 pounds is connected to a $\frac{1}{2}$-inch plate by $\frac{3}{4}$-inch rivets If the working stresses are 9,000 pounds per square inch in shear, and 18,000 pounds per square inch in bearing, how many rivets are required? Ans 11 rivets

4 The total stress transmitted by a member is 180,000 pounds The thickness of the member in bearing on a 4-inch pin is 1 inch, and the allowable intensity of bearing is 18,000 pounds per square inch (*a*) What is the required thickness of bearing? (*b*) What is the required thickness of pin plates? (*c*) What amount of stress is transmitted by the pin plates?

Ans $\begin{cases} (a) \ 2.5 \text{ in} \\ (b) \ 1.5 \text{ in} \\ (c) \ 108{,}000 \text{ lb.} \end{cases}$

5. The maximum bending moment on a pin is 157,100 inch-pounds, and the working stress in bending is 25,000 pounds per square inch. What is the required diameter of pin? Ans. 4 in

FLOOR SYSTEMS

FLOORS IN HIGHWAY BRIDGES

19. Kinds of Wearing Surface.—The top of the floor, commonly called the **wearing surface**, is, in highway bridges, usually composed of one or two layers of wooden plank supported by wooden or steel beams, called **stringers**,

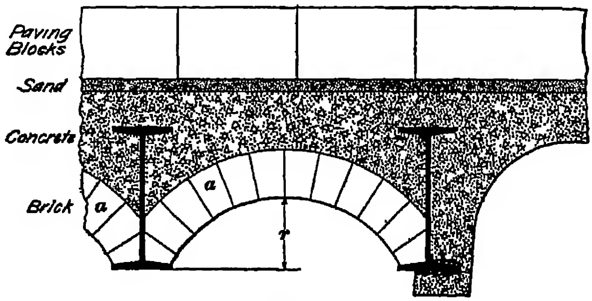


FIG 10

parallel to the girders or trusses and having a length equal to the panel length of the bridge. When a more durable floor is required, it is made of wooden or granite blocks, or

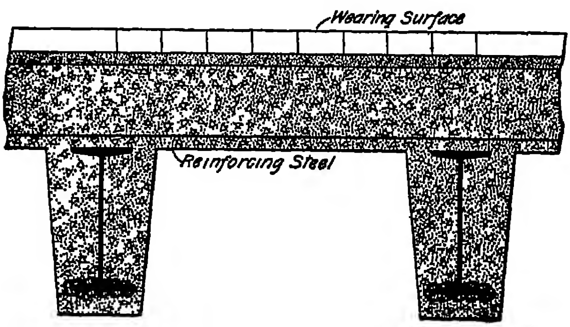


FIG 11

of asphalt; the wearing surface is then supported by brick or concrete arches between the stringers, as shown in Fig. 10; by reinforced-concrete slabs, as shown in Fig. 11;

or by buckle plates, as shown in Fig. 12. With each of these types of floors, concrete is brought up to the subgrade of the

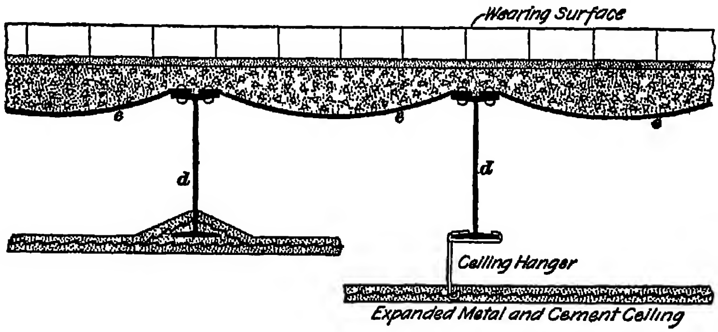


FIG 12

roadway, as shown, and the blocks or asphalt are placed on top with a layer of sand about 1 inch thick for a cushion coat.

20. Arches Between Beams.—When brick arches are used between the beams, as shown in Fig. 10, one ring of brick *a a* is put in, and concrete is filled in on top. The rise *r* of the arch depends wholly on the depth of stringer, being made greater for a deep than for a shallow stringer. The top of the bricks composing the arch should, in general, be no higher than the top of the stringers. The concrete should be at least 3 inches thick above the stringers The stringers are spaced about 3 feet apart in this type of floor. There is no necessity to calculate the strength of the arch just described, as it will have ample strength to carry any loads that pass over highway bridges if the stringers are spaced about 3 feet apart

In the most recent floors of this type, the ring of brick is omitted and the arch is formed of concrete The crown of the arch is usually kept below the top of the stringers and the concrete at the crown is made not less than about 6 inches in thickness This construction is shown at the right of Fig. 10. When this type of arch is used, it is customary to continue the concrete below and around the bottom flanges of the stringers for protection against

corrosion. For this purpose, expanded metal is bent so as to have it close to the top and bottom surfaces of the bottom flange, and the concrete is then put in to a thickness of not less than about 2 inches. This is a very serviceable type of floor, and is preferable in many ways to the brick arches.

21. Reinforced-Concrete Slab.—When the type of floor shown in Fig. 11 is used, the stringers are spaced about 3 feet apart, and the concrete slab is made from 6 to 9 inches thick. The amount of reinforcement depends on the load that the bridge is to carry. Ordinarily, not less than about 1 square inch cross-section of steel should be allowed to each foot of length of the slab measured at right angles to the steel rods. In this type of floor, it is customary to extend the concrete down around the stringers so as to protect the steel from corrosion. The method of doing this is shown in Fig. 11.

When reinforced concrete is used for floor slabs, great care should be taken in mixing and placing the concrete as well as in the design of the floor. Poor design and workmanship have been the cause of so many failures in reinforced-concrete construction in the last few years that it is evident that too much care cannot be taken.

22. Buckle Plates.—Fig. 12 shows the type of floor most frequently used when a solid floor is required. The stringers *d*, *d* support a thin plate *e*, on top of which concrete is placed; the wearing surface is placed on a sand cushion resting on the concrete. The plate *e* is originally flat, and square or very nearly square sections *f*, *f*, *f*, *f*, Fig. 13, are pressed outwards 2 or 3 inches, forming what are known as **buckles**. This plate can be had in sections about 3 feet wide and containing anywhere from one to eight buckles. The edges *g*, *g* of the plate, and short distances *h*, *h* between the buckles are made straight and flat so that they can be riveted to the supporting stringers. It has been found that these plates are much stronger when the edges are riveted down than when they are simply supported at the edges, and that they are much stronger when the buckle is turned

downwards, as shown, than when turned upwards in the form of an arch. The standard sizes of buckle plates vary from 2 ft. × 2 ft. 6 in. by intervals of about 3 inches to 4 ft. × 4 ft. 6 in. As a rule, it is best to have the buckles

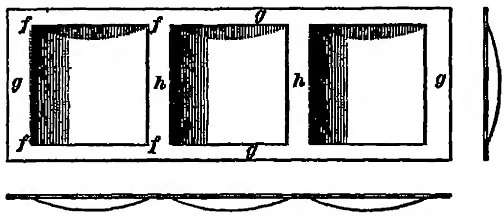


FIG 18

square or nearly square, and the distances between them about 4 inches; the edges should also be made about 4 inches wide.

Very little is known about the actual strength of buckle plates, but it is known that, if the stringers are spaced not more than 3 feet 6 inches apart, a thickness of $\frac{5}{16}$ inch is sufficient under the sidewalk and $\frac{3}{8}$ inch under the roadway to support any loads that will ever come on them in an ordinary highway bridge. The buckle plates should always be riveted at the edges to the stringers that support them, with $\frac{3}{4}$-inch rivets spaced about 5 inches apart.

When buckle plates are used in the floors of bridges that cross steam railroads, many pockets are formed by the stringers and floorbeams that extend down below the buckle plates. These pockets become filled with the gases that escape from the smokestacks of locomotives passing under them, and the gases rapidly corrode the steel. On account of trains passing under the bridge, it is extremely difficult to get at these corroded surfaces to clean and paint them. For this reason, it is the best practice, at the present time, to paint carefully and thoroughly all the exposed surfaces of steel under the floor as soon as the bridge is built, and then to construct a ceiling under the bottom flanges of the stringers and floorbeams. The ceiling is usually constructed of expanded metal and concrete 2 or 3 inches in thickness, and so arranged that no pockets are formed. This is

accomplished, in some cases, by supporting the ceiling directly on the bottom flanges of the stringers, as shown at the left of Fig. 12, and in others by placing the ceiling level with the bottom of the floorbeam and supporting the expanded metal by some form of ceiling hanger as shown at the right of Fig. 12.

FLOORS IN RAILROAD BRIDGES

23. Open Floors.—The standard open floor for railroad bridges consists of cross-ties resting on stringers, as shown in Fig. 14. The ties are usually 8 in. × 8 in. in cross-section, 10 feet long, and spaced about 12 inches center to center. The main rails *a*, *a* are spiked to the tops of the ties, between

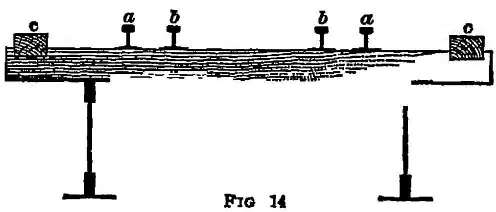


FIG 14

the main rails on which the cars run, two lines of rails *b*, *b* are spiked to the ties to act as guard-rails, their duty being to keep derailed cars from moving sidewise. Near the ends of the ties, guard timbers *c*, *c* are bolted to the tops of the ties to keep derailed cars from going over the side of the bridge and also to keep the ties properly spaced. The floor shown in Fig. 14 is the most economical type of floor, but it is not adapted to all conditions.

It is frequently necessary to adopt a special form of floor that occupies less room below the rail; besides, there are many cases in which it is objectionable to leave open the spaces between the ties. Various forms of solid floors are used to meet special conditions, and some of them will be considered here.

24. Trough Floors.—A style of **trough floor** used to a considerable extent in the past is shown in cross-section in Fig. 15. The **trough** *t*, *t* consists of an upper and a lower

half with inclined flanges riveted together as shown at *a*, *a*. The trough section is continued straight across from girder to girder or truss to truss, and the ends rest on shelf angles

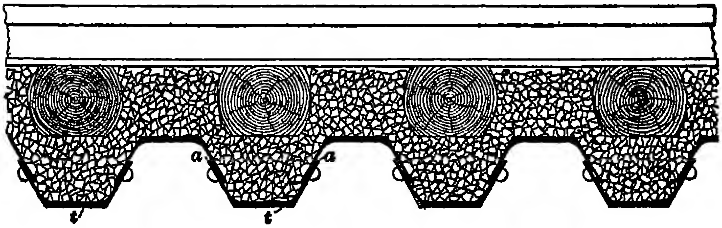


Fig. 15

or on the bottom flanges of the beams, as shown in Fig. 16. The top of the trough is covered with gravel or broken-stone ballast, and the ties are set directly on the ballast in the same way as on solid ground. This form of trough is being superseded by that shown in Fig. 17, which is stronger and can be connected much better to the girders or trusses that support it, in this type, the troughs are rectangular; they are composed of plates and angles, and can be given any desired strength. The ballast and ties occupy the same relative positions with respect to the trough as in Fig. 15. The top of the trough should not be allowed to come closer to the rail than about 2 inches.

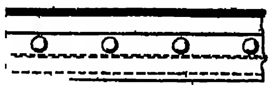


Fig 16

25. Solid Plate Floors. Another form of solid floor is made by placing a flat plate *a a*, $\frac{7}{16}$ or $\frac{1}{2}$ inch thick, on top of the beams or stringers, as shown in Fig. 18, and doing away with wooden ties entirely. The rail is placed directly

on top of the plate, and is held in place by small clips *c, c* that are bolted to the top plate. In this type of floor, angles are riveted to the top of the plate *a a,* they are called **guard angles** and serve the same purpose as guard-rails.

Up to the present time, none of the types of solid floors used in railroad bridges have given entire satisfaction. This

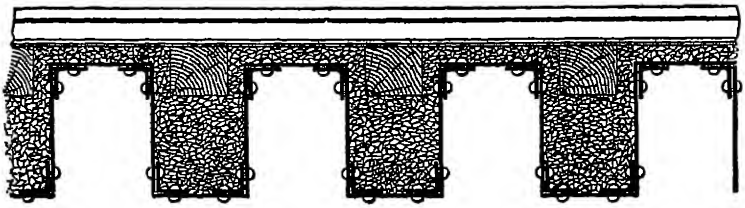


FIG. 17

is due principally to the fact that water and dirt accumulate in contact with the steel and the latter corrodes very rapidly. During the last few years, various forms of reinforced-concrete floors have been tried. These have almost all been modifications of the general type shown in Fig. 11 for highway bridges. The ballast is usually placed on top of the concrete, and a thickness of several inches placed under the ties.

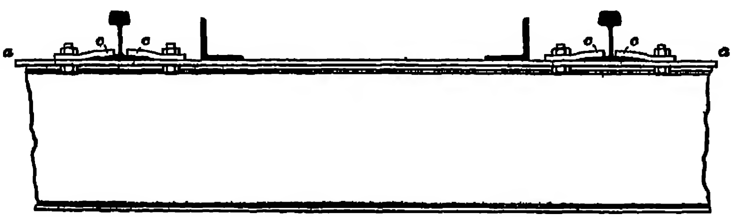


FIG 18

These floors have not been in use for a sufficient length of time to establish their suitability for this purpose. Some engineers have adopted them as standard for certain conditions; others will not use them on account of the liability of the concrete to crack, owing to the impact and vibration of trains and the deflection of the bridge.

STRINGERS AND STRINGER CONNECTIONS

26. Stringers.—Stringers of floor systems are usually either wooden beams, I beams, or plate girders. When plate girders are used, each flange is generally composed of two angles with no flange plates, although in very long spans flange plates are sometimes used. The design of stringers is no more difficult than that of simple beams, but the design of the connections should be made with care.

27. Stringer Connection Angles.—For the purpose of connecting the stringer to the floorbeam, connection angles are riveted to the ends of the stringers. Fig. 19 represents the side view (*a*) and end view (*b*) of the end of an I-beam stringer furnished with hitch, or connection, angles *c*. The hitch angles are made as long as possible, that is, as long as the straight portion of the web, and riveted tight to the web.

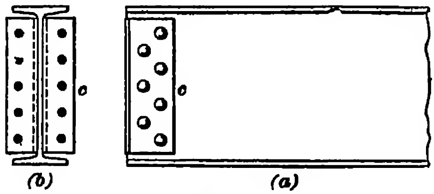


FIG 19

Fig. 20 represents the side view (*a*) and end view (*b*) of the end of a plate-girder stringer furnished with hitch or connection angles *c*. The hitch angles are made as long as possible, that is, very nearly as long as the clear distance between outstanding legs of the flange angles, and are riveted to the outside surfaces of the flange angles, the space between them and the web between the flange angles being filled by means of fillers *d* having the same thickness as the flange angles. These fillers are usually of greater width

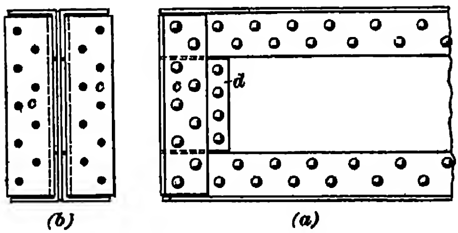


FIG. 20

than the connection angle, and are made tight by riveting them to the web outside of the leg of the connection angle, forming what are known as **tight fillers.**

The number of rivets required to connect the connection angles to the end of a stringer is found by dividing the maximum reaction on the stringer by the value of one rivet. For an I-beam stringer, the critical value of the rivet is usually its value in bearing on the web. For a plate-girder stringer with tight fillers, the combined thickness of web and tight fillers is counted as bearing on the rivet, and the critical value of the rivet is usually its value in double shear.

EXAMPLE.—The maximum reaction at the end of the I beam represented in Fig 19 is 45,000 pounds. How many $\frac{3}{4}$-inch rivets are required to connect the hitch angles to the web, which is $\frac{1}{2}$ inch in thickness, if the working stresses in shear and bearing are 9,000 and 18,000 pounds per square inch, respectively?

SOLUTION.—Consulting Table XXXIX, the value of a $\frac{3}{4}$-inch rivet in double shear is found to be 7,950 lb., and in bearing on a $\frac{1}{2}$-inch web, 6,750 lb. Since the latter is the smaller, it is the critical value of the rivet, and the required number of rivets is

$$45,000 \div 6,750 = 6.7, \text{ or, say, } 7. \quad \text{Ans.}$$

28. Stringer Connection to Floorbeams.—There are several methods of connecting stringers to floorbeams. The

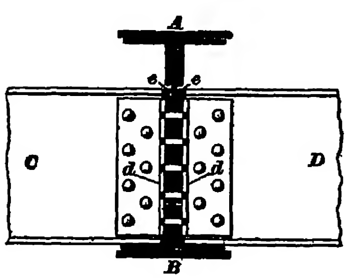


FIG 21

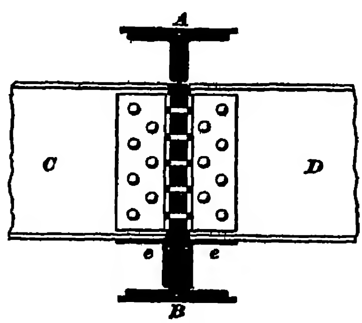


FIG 22

principal methods are represented in Figs. 21 to 24, in all of which AB is the cross-section of the floorbeam, and CD are portions of the stringers that connect to them. Fig. 21 shows the connection that is employed when it is desirable to use

stringers and it is necessary to keep the depths of floor as small as possible. The bottoms of the stringers rest directly on top of the bottom flange of the floorbeam. The outstanding legs *d* of the connection angles come in contact with the vertical legs of the bottom flange angles of the floorbeams, and fillers *e* having the same thickness as those angles are placed between the connection angles and the web. This connection can be used only when the depth of the floorbeam is but little greater than that of the stringer.

Figs. 22 and 23 show connections that are sometimes used when the floorbeam is considerably deeper than the stringer

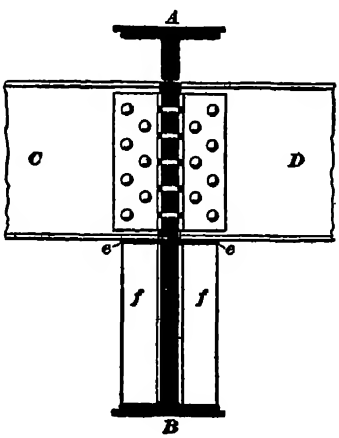


FIG 23

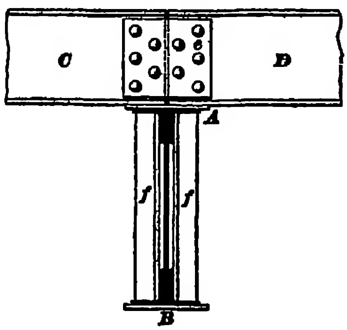


FIG. 24

and the bottom of the stringer is a sufficient distance above the bottom of the floorbeam. A chair or shelf is provided by riveting to the bottom flange or web of the floorbeam a short shelf angle *e* parallel to the flange; and when this is above the bottom flange, as in Fig. 23, inserting short stiffeners *f* between the outstanding legs of the shelf and the bottom flange angles. The connection angles are riveted to the floorbeam web the same as if the chair or shelf were not provided; the latter simply affords additional strength and better distributes the load from the stringer over the floorbeam. The connection shown in Fig. 23 is by far the

best type of connection between stringers and floorbeam, and should be used wherever conditions permit. The designer should endeavor so to arrange his connections that this form can be used.

In deck truss bridges and in through bridges in which the allowable depth of floor is comparatively great, the stringers are sometimes placed directly on top of the top flanges of the floorbeams, as represented in Fig 24. When they are placed in this position, the ends of the stringers are usually connected by plates *e* that serve to keep them in line. Stiffeners *f* having a length of at least one-half the depth of floorbeam and preferably the full depth of the floorbeam should be placed under each stringer.

29. It is very important that there should be sufficient rivets to connect the connection angles to the floorbeams; as it has been found, in practice, that under the actual conditions of loading these rivets become loose before any others in the bridge, showing that they are subjected to secondary stresses whose magnitude cannot be calculated. The number of rivets required is found in the ordinary way, by dividing the load coming from the stringer by the value of one rivet. The critical value of one of these rivets connecting the angles to the floorbeam is usually the value in single shear.

The design of stringers and stringer connections will be treated in subsequent Sections.

FLOORBEAMS

30. Shapes Used for Floorbeams.—I beams are sometimes used for floorbeams of highway bridges, but in railroad bridges, and frequently in highway bridges, plate girders are used.

31. Floorbeam Connections.—Floorbeams are connected to girders and trusses in almost the same way as stringers are connected to floorbeams, but the actual connection depends on the details in each case.

LATERAL SYSTEMS

32. Lateral Trusses.—The lateral trusses of truss bridges were discussed in *Stresses in Bridge Trusses*, Part 5. The lateral trusses of plate-girder bridges are somewhat different from those of truss bridges, but the same principles apply to both. *It is customary to ignore the stresses due to wind pressure and centrifugal force in the girders when the latter are considered as chord members of lateral trusses.*

The diagonals of lateral trusses are usually called **laterals.**

33. In deck girder bridges, lateral trusses are provided in both the top and bottom flanges. Each has the form represented in Fig. 25. The stresses are found in the same

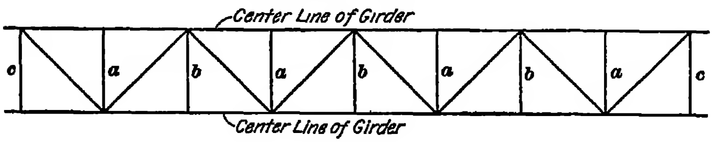


FIG 25

way as in the Warren truss. As there is only one system of diagonals, the stresses in them will be sometimes tension and sometimes compression, according to the direction in which the wind blows, and they must be designed for this reversal of stress according to the rules given in *Bridge Specifications*. In Fig. 25, the members marked *a* are simply cross-struts to connect the two flanges; those marked *b* are intermediate transverse frames; and those marked *c* are end transverse frames It is customary to locate the frames so that the diagonals will make an angle of about 45° with the center line of the bridge.

34. In half-through plate-girder bridges, there is but one lateral truss, composed of two systems of diagonals like those described for truss bridges. When the panel length of the bridge is less than one-half the distance center to center of girders, the panels of the lateral trusses are frequently made twice the length of the panel of the bridge, as

represented in Fig. 26 In this figure, a, a are the end floorbeams, and b, c, the intermediate. The diagonals of the two systems intersect each other at the center of the floorbeams marked b, and are attached to each other and to the floorbeams by means of connection plates and angles. The ends of the laterals intersect near the center lines of the girders, at the ends of the floorbeams marked c, and are connected to each other, to the floorbeams, and to the girders by means of connection plates. It is impossible to calculate accurately

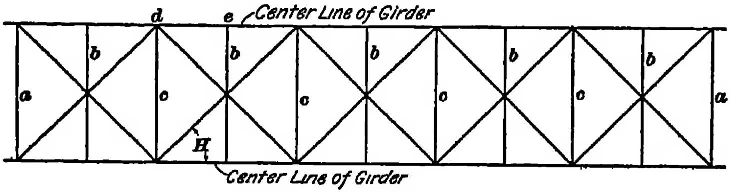


FIG 26

the stresses in the diagonals of this type of truss: the usual practice is to assume that the shear in any panel of the bridge due to wind pressure or centrifugal force or both is resisted by one diagonal in tension, the other being out of action. For example, if there is no centrifugal force acting on the bridge represented in Fig 26, and the maximum shear in the panel de due to wind pressure is V_w, then, as the wind may blow in either direction, both diagonals must be designed for a tension equal to $V_w \csc H$.

35. Lateral Connections.—The method of connecting laterals to trusses, girders, and floorbeams depends to a great extent on the connections of stringers to floorbeams and floorbeams to girders and trusses. When the floorbeams of half-through bridges rest on the bottom flanges of the girders, as is frequently the case, the lateral connection is invariably made as represented in Fig. 27. In this figure, $a\,a$ is a portion of the lower flange of the plate girder; $b\,b$, the lateral connection plate resting on top of the flange angles; $c\,c$, a portion of the bottom flange of the floorbeam; $d\,d$, the bottom flanges of the stringers; and $e\,e$, the laterals. When the stringers rest directly on top of the bottom flange

angle of the floorbeam, the laterals are placed below the plates, and fillers *f* of sufficient thickness are placed between the bottom of the stringer and the top of the lateral, as represented at the left of the floorbeam in Fig. 27. When the stringers are riveted to the floorbeams, a short distance, usually not over 7 inches, above the bottom flange angles, the laterals are placed above the connection plates and are connected to the stringers by means of lug or hitch angles *g*,

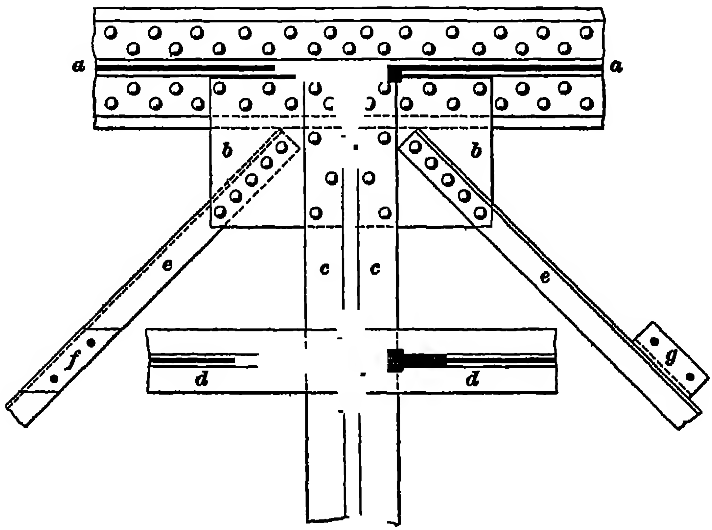


Fig 27

as represented at the right of the floorbeam in Fig. 27. When the bottoms of stringers are more than 6 or 7 inches above the bottoms of the floorbeams, the connections between the laterals and tho stringers are sometimes dispensed with, and the laterals are located in either of the two ways represented in Fig. 27, the fillers *f* and hitch angles *g* being omitted.

36. When the bottoms of the stringers are more than 6 or 7 inches above the bottoms of the floorbeams, and it is desired to maintain the connection between the stringers and the laterals, the arrangement represented in Fig. 28 is

used. The angles *a*, *a* are connection or hitch angles riveted to the webs of the girder and floorbeam above the lower flange angles; *b*, *b* are the lateral connection plates, which may be either above or below the angles *a*, *a*; and *c*, *c* are the laterals, which are invariably placed below the connection plates, as shown, and at such an elevation that they can be riveted directly to the lower flanges of the stringers, as

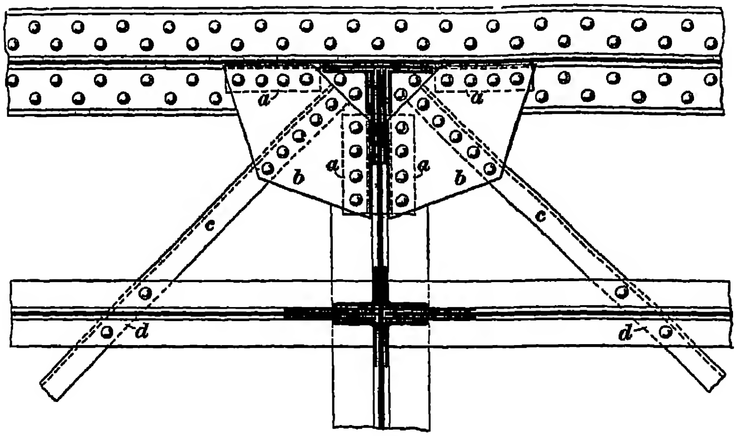


FIG 28

at *d*, *d*, without the necessity of any intermediate filling pieces. There should, as a rule, never be less than four rivets in the connection of the lateral connection plate *b* to either hitch angle *a*, nor less than five rivets in the connection of any hitch angle to the web of the girder or floorbeam.

37. The lateral connections of deck bridges are similar to those of half-through bridges. The connection plates are either riveted to the flanges, as represented in Fig. 27, or to hitch angles that are riveted to the webs of the girders below the top flange angles and above the bottom flange angles, as represented in Fig. 28 The latter method is to be preferred in the case of a deck bridge, because it lowers the top lateral system so that it does not interfere with the wooden-floor system, as sometimes happens when the lateral connection

plates are riveted directly to the outstanding legs of the top flange angles.

38. Transverse Frames.—In both of the cases mentioned in the preceding article, a pair of stiffeners is located on the girder at each panel point of the lateral truss, and a transverse strut or frame is riveted to the stiffeners Fig. 29 represents a cross-section of a deck-girder bridge and an end

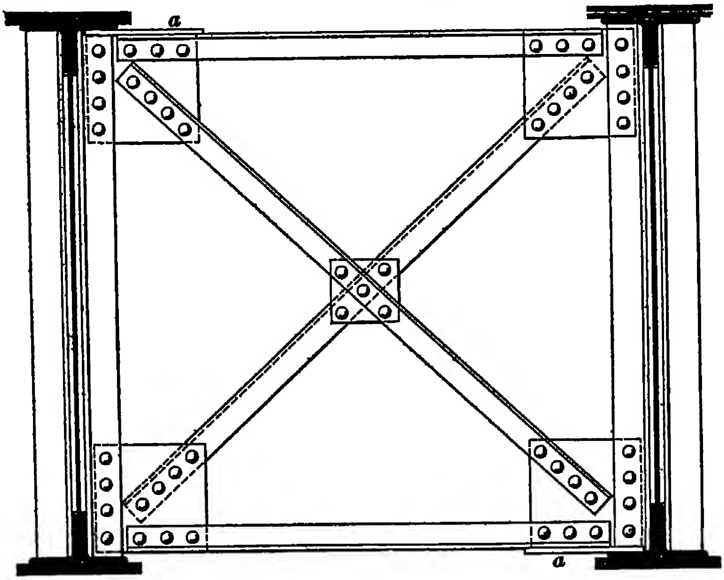


FIG 29

transverse frame. The lateral connection plates *a*, *a* are riveted directly to the outstanding legs of the flange angles, and the frame is made deep enough to fit between the connection plates. Fig. 30 also represents a cross-section of a deck plate-girder bridge and an end transverse frame. The lateral connection plates in this case are riveted to the hitch angles, and the frame is made deep enough to fit between the connection plates. In both of these figures, the same detail is used for the intermediate transverse frames as for the end frames, with the exception that but one diagonal is provided. Where only a transverse strut is desired, both diagonals are omitted.

When deck-plate girders are very shallow, say less than 3 feet deep, the transverse frames are sometimes composed of a solid web and two angles, as represented in Fig. 31. or

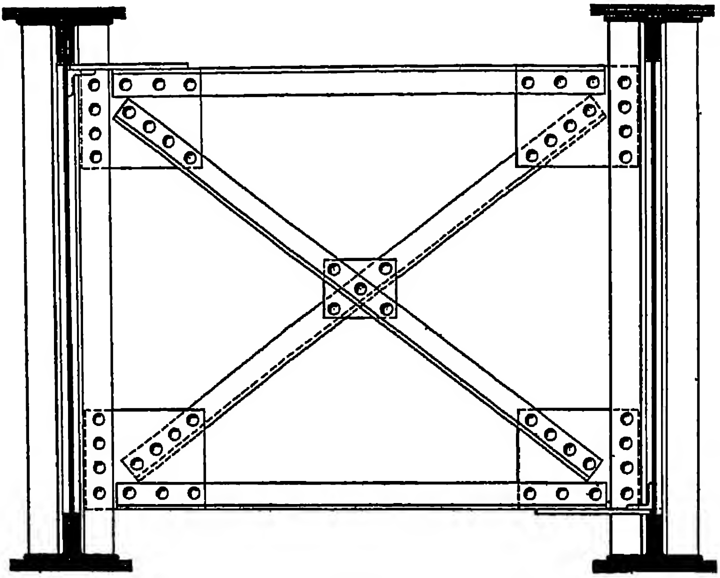


FIG 30

are latticed, as represented in Fig. 32; in this case the lower lateral truss is usually omitted. When these types of trans-

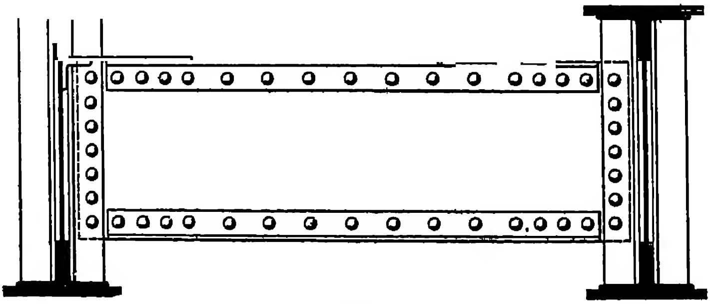


FIG 31

verse frames are used, the end and intermediate frames are made the same. The frames represented in Figs. 31 and 32

are used also to connect the ends of end stringers to each other and to the girders or trusses when there are no end floorbeams.

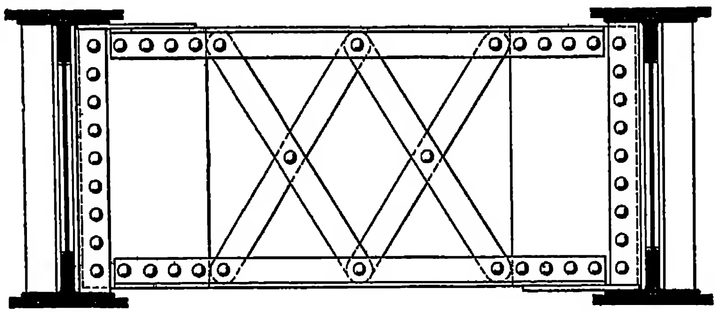


FIG 32

39. Portals and Transverse Struts.—Portals and transverse struts and frames of through bridges are arranged to conform to other details, and will be treated in connection with design.

BEARINGS

BEDPLATES AND ROCKERS

40. Bedplates.—Where trusses or girders rest on the bridge seat or abutment, large steel plates, called **bedplates,** are placed on the masonry so as to distribute the pressure evenly. The required area of bearing is found by dividing the reaction by the allowable intensity of pressure. In short-span bridges, say less than 75 feet, the reaction is sufficiently distributed by bedplates $\frac{3}{4}$ or 1 inch thick.

41. Rockers.—For spans longer than 75 feet, the method of distribution given in Art. 40 is unsatisfactory, as the greater deflection of the span makes the contact between the bedplate and masonry somewhat imperfect, throwing more load on the front than on the back of the bedplate. To distribute the load more evenly over the masonry, **rocker bearings** are used. A rocker bearing is represented in

outline in Fig. 33. The girder or truss rests on the upper half *a* of a **pedestal**, in the bottom of which is a semicylindrical hole that just fits a pin placed at right angles to the truss or girder. This pin is supported by the lower half *b* of the pedestal, which rests on the masonry. When this device

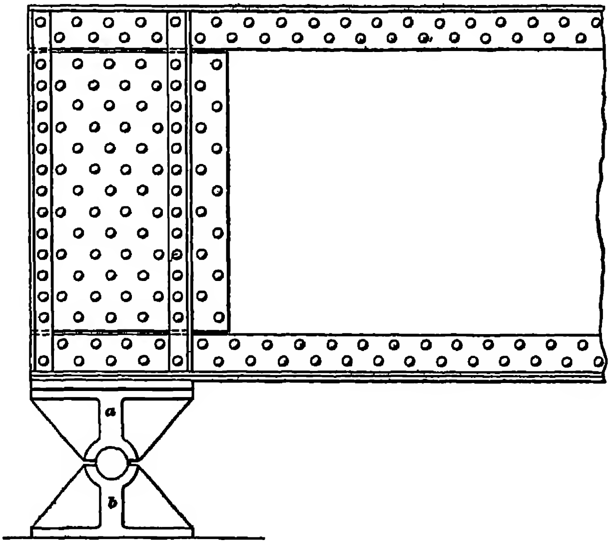


FIG 33

is used, the load is always concentrated on the pin, and the upper half rocks slightly back and forth as the truss or girder deflects; the load being always concentrated on the pin, which is at the center of the pedestal, the pressure on the masonry is always evenly distributed.

ROLLERS

42. Changes of temperature cause bridges to expand and contract. The difference between the lengths of the bridge at the highest and lowest temperatures may amount to 1 inch in 100 feet For spans less than about 75 feet, it is assumed that the bridge will adjust itself to these changes, and the top of the bedplate and bottom of the bridge at one end

(commonly called the **expansion end**) are planed smooth so that the end can slide back and forth more easily. For spans of greater length, the load is so great that the ends slide with difficulty, and it is necessary to place rollers, called **expansion rollers**, under one end, and a bedplate between the rollers and the masonry, as shown in Fig. 34. When the

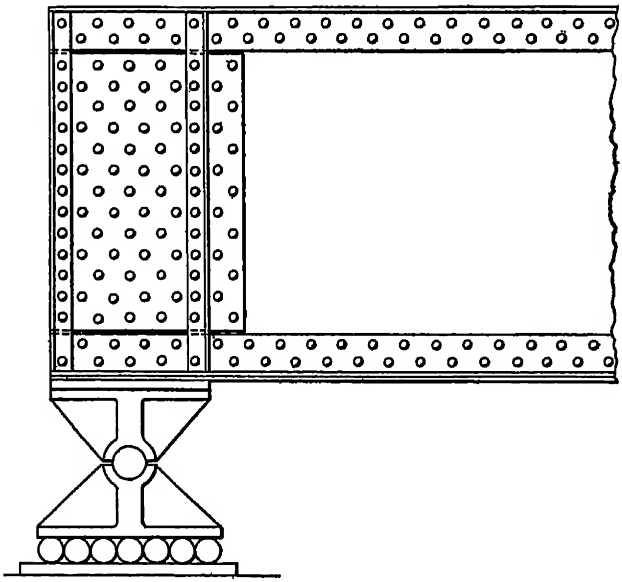


FIG 34

truss expands or contracts, the end under which the rollers are located moves back and forth on the rollers. These rollers are usually made from 3 to 6 inches in diameter.

The design of pedestals and rollers will be treated elsewhere.

ANCHOR BOLTS

43. Both ends of every truss or girder are fastened to the masonry by means of bolts, called **anchor bolts.** Fig. 35 shows four kinds of anchor bolts that are commonly used. For each of these, a hole about 12 inches deep is drilled in the masonry, the bolt is dropped into this hole,

and the rest of the hole is then filled with sulphur, lead, or cement. The bolts shown at *a* and *b* are sometimes called **rag or dog bolts**; that shown at *c* is simply a screw bolt. In the form shown at *d*, a wedge is inserted in the end of the bolt, and this end is placed in the hole, the bolt is then struck hard on top to force out the sides at the bottom, so that they will grip the masonry firmly. The rest of the hole is then filled with sulphur, lead, or cement.

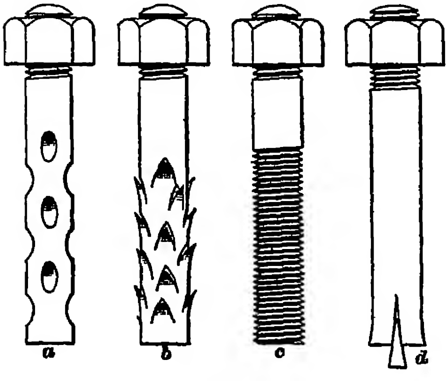


FIG 35

At the expansion end of the bridge, the holes in the pedestal or bottom flange through which the anchor bolts must pass are made longer than the width, as shown in Fig. 36, so that that end can move freely backwards and forwards. The bolts at this end simply serve to keep the bridge from moving sidewise; those at the other end, commonly called the **fixed end**, serve to hold the bridge in place longitudinally as well as laterally.

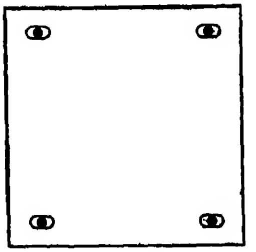


FIG 36

CAMBER

44. When a bridge is loaded, it deflects and forms a curve, the center of the span being lower than the ends. To counteract this tendency, and make the floor nearly level when the bridge is loaded, trusses are built so that they curve slightly upwards when there is no load on the bridge, and assume a position almost horizontal when the load is applied. This upward curve is spoken of as the **camber**.

Trusses are cambered by making the panels of the top chord slightly longer than the panel lengths of the lower chord, the usual allowance being $\frac{1}{8}$ inch for every 10 feet. For example, if the panel length of a truss is 20 feet, the pins or intersections of the center lines in the top chord will be 20 feet $\frac{1}{4}$ inch apart. This gives the truss the appearance shown in Fig 37 to an exaggerated vertical scale. The joints in the chords are approximately on curves, and the verticals may be considered to be on radii to these curves.

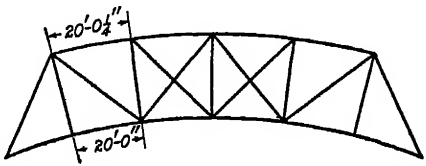


FIG. 37

The actual length of a diagonal of a truss with camber is equal to the length of a diagonal of a truss of the same depth having no camber, and whose panel length is a mean between the actual lengths of the top and bottom chord members. For example, if the height of truss in Fig. 37 is 20 feet, the mean of the lengths of the two chord members in one panel is

$$\frac{1}{2} \times (20 \text{ feet} + 20 \text{ feet } \tfrac{1}{4} \text{ inch}) = 20 \text{ feet } \tfrac{1}{8} \text{ inch}$$

and the actual length of a diagonal is

$$\sqrt{(20 \text{ feet})^2 + (20 \text{ feet } \tfrac{1}{8} \text{ inch})^2} = 28.292 \text{ feet}$$

INDEX TO BRIDGE TABLES

BRIDGE TABLES

TABLE I

AREAS AND WEIGHTS OF ROUND RODS

Diameter Inches	Area of Section Square Inches	Weight per Foot Pounds	Diameter Inches	Area of Section Square Inches	Weight per Foot Pounds	Diameter Inches	Area of Section Square Inches	Weight per Foot Pounds
1/16	003	010	1 9/16	1 92	6 52	3 1/8	7 67	26 08
1/8	012	042	1 5/8	2 07	7 05	3 1/4	8 30	28 20
3/16	028	094	1 11/16	2 24	7 60	3 3/8	8 95	30 42
1/4	.049	167	1 3/4	2 41	8 18	3 1/2	9 62	32 71
5/16	.077	261	1 13/16	2 58	8 77	3 5/8	10 32	35 09
3/8	110	375	1 7/8	2 76	9 39	3 3/4	11 04	37 56
7/16	150	511	1 15/16	2 95	10 02	3 7/8	11 79	40 10
1/2	.196	667	2	3 14	10 68	4	12 57	42 73
9/16	248	845	2 1/16	3 34	11 36	4 1/8	13 36	45 44
5/8	307	1 04	2 1/8	3 55	12 06	4 1/4	14 19	48 24
11/16	371	1 26	2 3/16	3 76	12 78	4 3/8	15 03	51 11
3/4	442	1 50	2 1/4	3 98	13 52	4 1/2	15 90	54 07
13/16	518	1 76	2 5/16	4 20	14 28	4 5/8	16 80	57 12
7/8	601	2 04	2 3/8	4 43	15 07	4 3/4	17 72	60 25
15/16	690	2 35	2 7/16	4 67	15 86	4 7/8	18 66	63 46
1	785	2 67	2 1/2	4 91	16 69	5	19 63	66 76
1 1/16	887	3 01	2 9/16	5 16	17 53	5 1/8	20 63	70 14
1 1/8	994	3 38	2 5/8	5 41	18 40	5 1/4	21 65	73 60
1 3/16	1 11	3 77	2 11/16	5 67	19 29	5 3/8	22 69	77.15
1 1/4	1 23	4 17	2 3/4	5 94	20 20	5 1/2	23 76	80 77
1 5/16	1 35	4 60	2 13/16	6 21	21.12	5 5/8	24 85	84.49
1 3/8	1 48	5 05	2 7/8	6 49	22 07	5 3/4	25 97	88 29
1 7/16	1 62	5 52	2 15/16	6 78	23 04	5 7/8	27 11	92 17
1 1/2	1 77	6 01	3	7 07	24 03	6	28 27	96 14

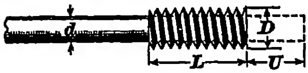
TABLE II

AREAS AND WEIGHTS OF SQUARE RODS

Side of Square Inches	Area of Section Square Inches	Weight per Foot Pounds	Side of Square Inches	Area of Section Square Inches	Weight per Foot Pounds	Side of Square Inches	Area of Section Square Inches	Weight per Foot Pounds
1/16	004	013	1 9/16	2 44	8 30	3 1/8	9 77	33 20
1/8	016	053	1 5/8	2 64	8 98	3 1/4	10 56	35 92
3/16	035	120	1 11/16	2 85	9 68	3 3/8	11 39	38 73
1/4	062	213	1 3/4	3 06	10 41	3 1/2	12 25	41 65
5/16	098	332	1 13/16	3 29	11 17	3 5/8	13 14	44 68
3/8	141	478	1 7/8	3 52	11 95	3 3/4	14 06	47 82
7/16	191	651	1 15/16	3 75	12 76	3 7/8	15 02	51 05
1/2	250	850	2	4 00	13 60	4	16 00	54 40
9/16	316	1 08	2 1/16	4 25	14 46	4 1/8	17 02	57 85
5/8	391	1 33	2 1/8	4 52	15 35	4 1/4	18 06	61 41
11/16	473	1 61	2 3/16	4 79	16 27	4 3/8	19 14	65 08
3/4	562	1 91	2 1/4	5 06	17 22	4 1/2	20 25	68 85
13/16	660	2 25	2 5/16	5 35	18 19	4 5/8	21 39	72 73
7/8	766	2 60	2 3/8	5 64	19 18	4 3/4	22 56	76 71
15/16	879	2 99	2 7/16	5 94	20 20	4 7/8	23 77	80 81
1	1 00	3 40	2 1/2	6 25	21 25	5	25 00	85 00
1 1/16	1 13	3 84	2 9/16	6 57	22 33	5 1/8	26 27	89 30
1 1/8	1 27	4.30	2 5/8	6 89	23 43	5 1/4	27 56	93 72
1 3/16	1 41	4 80	2 11/16	7 22	24 56	5 3/8	28 89	98 23
1 1/4	1 56	5 31	2 3/4	7 56	25 71	5 1/2	30 25	102 8
1 5/16	1 72	5 86	2 13/16	7 91	26 90	5 5/8	31 64	107 6
1 3/8	1 89	6 43	2 7/8	8 27	28 10	5 3/4	33 06	112 4
1 7/16	2 07	7 03	2 15/16	8 63	29 34	5 7/8	34 52	117 4
1 1/2	2 25	7 65	3	9 00	30 60	6	36 00	122 4

TABLE III

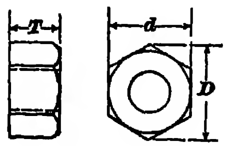
UPSET SCREW ENDS FOR ROUND AND SQUARE RODS



Round Rods				Square Rods			
Diameter of Rod Inches d	Diameter of Screw Inches D	Length of Screw Inches L	Additional Length to Form Upset Inches U	Side of Square Inches d	Diameter of Screw Inches D	Length of Screw Inches L	Additional Length to Form Upset Inches U
$\frac{3}{4}$	1	4	$3\frac{7}{8}$	$\frac{3}{4}$	$1\frac{1}{8}$	4	$3\frac{1}{2}$
$\frac{7}{8}$	$1\frac{1}{4}$	4	5	$\frac{7}{8}$	$1\frac{1}{4}$	4	4
1	$1\frac{3}{8}$	4	$4\frac{3}{8}$	1	$1\frac{1}{2}$	4	4
$1\frac{1}{8}$	$1\frac{1}{2}$	4	$3\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{5}{8}$	$4\frac{1}{2}$	$4\frac{1}{2}$
$1\frac{1}{4}$	$1\frac{5}{8}$	$4\frac{1}{2}$	$3\frac{7}{8}$	$1\frac{1}{4}$	$1\frac{7}{8}$	$4\frac{1}{2}$	$4\frac{1}{2}$
$1\frac{3}{8}$	$1\frac{3}{4}$	$4\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{3}{8}$	2	5	$4\frac{1}{8}$
$1\frac{1}{2}$	2	5	$4\frac{5}{8}$	$1\frac{1}{2}$	$2\frac{1}{4}$	5	$4\frac{3}{4}$
$1\frac{5}{8}$	$2\frac{1}{8}$	5	$4\frac{1}{4}$	$1\frac{5}{8}$	$2\frac{3}{8}$	$5\frac{1}{2}$	$4\frac{5}{8}$
$1\frac{3}{4}$	$2\frac{1}{4}$	5	4	$1\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{1}{2}$	$4\frac{1}{4}$
$1\frac{7}{8}$	$2\frac{3}{8}$	$5\frac{1}{2}$	$4\frac{1}{8}$	$1\frac{7}{8}$	$2\frac{3}{4}$	6	$5\frac{1}{8}$
2	$2\frac{1}{2}$	$5\frac{1}{2}$	$3\frac{7}{8}$	2	$2\frac{7}{8}$	6	$4\frac{3}{4}$
$2\frac{1}{8}$	$2\frac{5}{8}$	$5\frac{1}{2}$	$3\frac{5}{8}$	$2\frac{1}{8}$	3	6	$4\frac{3}{8}$
$2\frac{1}{4}$	$2\frac{7}{8}$	6	$4\frac{5}{8}$	$2\frac{1}{4}$	$3\frac{1}{4}$	$6\frac{1}{2}$	$5\frac{1}{8}$
$2\frac{3}{8}$	3	6	$4\frac{3}{8}$	$2\frac{3}{8}$	$3\frac{1}{2}$	7	$6\frac{1}{8}$
$2\frac{1}{2}$	$3\frac{1}{8}$	$6\frac{1}{2}$	$4\frac{3}{8}$	$2\frac{1}{2}$	$3\frac{5}{8}$	8	$6\frac{1}{4}$
$2\frac{5}{8}$	$3\frac{1}{4}$	$6\frac{1}{2}$	$4\frac{1}{4}$	$2\frac{5}{8}$	$3\frac{7}{8}$	8	$6\frac{3}{4}$
$2\frac{3}{4}$	$3\frac{3}{8}$	7	$4\frac{1}{4}$	$2\frac{3}{4}$	4	8	6
$2\frac{7}{8}$	$3\frac{5}{8}$	8	$5\frac{1}{2}$	$2\frac{7}{8}$	$4\frac{1}{4}$	9	8
3	$3\frac{3}{4}$	8	$5\frac{1}{4}$	3	$4\frac{1}{2}$	9	$7\frac{1}{2}$

TABLE IV

SIZES AND WEIGHTS OF HEXAGON NUTS



Diameter of Screw Inches	Thickness of Nut T Inches	Short Diameter d Inches	Long Diameter D Inches	Weight of 100 Nuts Pounds
½	½	1	1.15	9.8
⅝	¾	1¼	1.44	22.9
¾	⅞	1½	1.73	39
⅞	1	1⅝	1.88	50
1	1⅛	1¾	2.02	64
1⅛	1¼	2	2.31	96
1¼	1⅜	2¼	2.60	134
1⅜	1½	2½	2.89	180
1½	1⅝	2¾	3.18	235
1⅝	1¾	3	3.46	300
1¾	1⅞	3¼	3.75	370
1⅞	2	3½	4.04	460
2	2	3½	4.04	450
2¼	2¼	3¾	4.33	560
2½	2½	4¼	4.91	810
2¾	2¾	4½	5.20	980
3	3	4¾	5.48	1,150
3¼	3¼	5	5.77	1,340
3½	3½	5¼	6.06	1,580

TABLE V

APPROXIMATE MAXIMUM LENGTHS OF STEEL PLATES, IN FEET

Width Inches	Thickness, in Inches									
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
10	45	70	70	70	70	70	70	70	70	70
20	45	90	90	80	80	80	75	65	60	50
30	*	90	90	80	60	60	50	40	40	30
40	*	80	90	80	65	50	40	35	30	25
50	28	40	40	40	34	30	28	25	22	22
60	25	38	40	33	30	26	24	22	18	18
70	22	35	34	29	26	22	20	19	17	16
80	20	30	30	25	23	19	19	17	15	15
90	13	27	27	21	18	17	16	14	13	13
100	8	16	20	18	16	15	14	12	11	11
110	*	*	14	13	13	11	10	10	9	*
120	*	*	*	10	10	10	10	10	*	*

*Plates having these dimensions are not ordinarily handled by the rolling mills.

TABLE VI

AREAS OF SECTIONS OF STEEL PLATES, IN SQUARE INCHES

Width Inches	Thickness, in Inches							
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$
$\frac{1}{4}$	0156	0313	0469	0625	0781	0938	1094	1250
$\frac{1}{2}$	0313	0625	0938	1250	1563	1875	2188	2500
$\frac{3}{4}$	0469	0938	1406	1875	2344	2813	3281	3750
1	0625	1250	1875	2500	3125	3750	4375	5000
2	1250	2500	3750	5000	6250	7500	8750	1 000
3	1875	3750	5625	750	9375	1 125	1 313	1 500
4	2500	5000	7500	1 000	1 250	1 500	1 750	2 000
5	3125	6250	9375	1 250	1 563	1 875	2 188	2 500
6	3750	7500	1 125	1 500	1 875	2 250	2 625	3 000
7	4375	8750	1 313	1 750	2 188	2 625	3 063	3 500
8	5000	1 000	1 500	2 000	2 500	3 000	3 500	4 000
9	5625	1 125	1 688	2 250	2 813	3 375	3 938	4 500
10	6250	1 250	1 875	2 500	3 125	3 750	4 375	5 000
11	6875	1 375	2 063	2 750	3 438	4 125	4 813	5 500
12	7500	1 500	2 250	3 000	3 750	4 500	5 250	6 000
13	8125	1 625	2 438	3 250	4 063	4 875	5 688	6 500
14	8750	1 750	2 625	3 500	4 375	5 250	6 125	7 000
15	9375	1 875	2 813	3 750	4 688	5 625	6 563	7 500
16	1 000	2 000	3 000	4 000	5 000	6 000	7 000	8 000
18	1 125	2 250	3 375	4 500	5 625	6 750	7 875	9 000
20	1 250	2 500	3 750	5 000	6 250	7 500	8 750	10 000
30	1 875	3 750	5 625	7 500	9 375	11 250	13 125	15.000
40	2 500	5 000	7 500	10 000	12 500	15 000	17 500	20.000
50	3 125	6 250	9 375	12 500	15 625	18 750	21 875	25 000
60	3 750	7 500	11 250	15 000	18 750	22 500	26 250	30 000
70	4.375	8 750	13 125	17 500	21 875	26 250	30 625	35 000
80	5 000	10 000	15 000	20 000	25 000	30 000	35 000	40 000
90	5 625	11 250	16 875	22 500	28 125	33 750	39 375	45 000
100	6 250	12 500	18 750	25 000	31 250	37 500	43 750	50 000

TABLE VI—(*Continued*)

AREAS OF SECTIONS OF STEEL PLATES, IN SQUARE INCHES

Width Inches	Thickness, in Inches							
	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
¼	1406	1563	1719	1875	2031	2188	2344	250
½	2813	3125	.3438	3750	4063	4375	4688	500
¾	4219	4688	5156	5625	6094	6563	7031	750
1	5625	6250	6875	7500	8125	8750	9375	1 000
2	1 125	1 250	1 375	1 500	1 625	1 750	1 875	2 000
3	1 688	1 875	2 063	2 250	2 438	2 625	2 813	3 000
4	2 250	2 500	2 750	3 000	3 250	3 500	3 750	4 000
5	2 813	3 125	3 438	3 750	4 063	4 375	4 688	5 000
6	3 375	3 750	4 125	4 500	4 875	5 250	5 625	6 000
7	3 938	4 375	4 813	5 250	5.688	6 125	6 563	7 000
8	4 500	5 000	5 500	6 000	6 500	7 000	7 500	8 000
9	5 063	5 625	6 188	6 750	7 313	7 875	8 438	9 000
10	5 625	6 250	6 875	7 500	8 125	8 750	9 375	10 000
11	6.188	6 875	7 563	8 250	8 938	9 625	10 313	11 000
12	6 750	7 500	8 250	9 000	9 750	10 500	11 250	12 000
13	7 313	8 125	8 938	9 750	10 563	11 375	12 188	13.000
14	7 875	8 750	9 625	10 500	11 375	12 250	13 125	14 000
15	8 438	9 375	10 313	11 250	12 188	13 125	14 063	15 000
16	9 000	10 000	11 000	12 000	13 000	14 000	15 000	16 000
18	10 125	11 250	12 375	13 500	14 625	15 750	16 880	18 000
20	11 250	12 500	13 750	15 000	16 250	17 500	18 750	20 000
30	16 875	18 750	20 625	22 500	24 375	26 250	28 125	30 000
40	22 500	25 000	27 500	30 000	32.500	35 000	37 500	40 000
50	28 125	31 250	34 375	37 500	40 625	43 750	46 875	50 000
60	33 750	37 500	41 250	45 000	48 750	52 500	56 250	60 000
70	39 375	43 750	48 125	52 500	56 875	61 250	65 625	70 000
80	45 000	50 000	55 000	60 000	65 000	70 000	75 000	80 000
90	50 625	56 250	61 875	67 500	73.125	78 750	84 375	90 000
100	56 250	62 500	68 750	75 000	81 250	87 500	93 750	100 000

TABLE VII

WEIGHT OF STEEL PLATES, IN POUNDS PER LINEAR FOOT

Width Inches	Thickness, in Inches							
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$
$\frac{1}{4}$	0531	1063	.1594	2125	2656	3188	3719	4250
$\frac{1}{2}$	1063	2125	3188	4250	5313	6375	7438	8500
$\frac{3}{4}$	1594	3188	4781	6375	7969	9563	1 116	1 275
1	2125	4250	6375	8500	1 063	1 275	1.488	1 700
2	4250	850	1 275	1 700	2 125	2 550	2 975	3 400
3	6375	1 275	1 913	2 550	3 188	3 825	4 463	5 100
4	8500	1 700	2 550	3 400	4 250	5 100	5 950	6 800
5	1 063	2 125	3 188	4 250	5 313	6 375	7 438	8 500
6	1 275	2 550	3 825	5 100	6 375	7 650	8 925	10 200
7	1 488	2 975	4 463	5 950	7 438	8 925	10 413	11 900
8	1 700	3 400	5 100	6 800	8 500	10 200	11 900	13.600
9	1 913	3 825	5 738	7 650	9 563	11 480	13.390	15 300
10	2.125	4 250	6 375	8 500	10 630	12 750	14 880	17 000
11	2 338	4 675	7 013	9 350	11 690	14 030	16 360	18 700
12	2 550	5 100	7 650	10 200	12 750	15 300	17 850	20 400
13	2 763	5 525	8 288	11 050	13 810	16.580	19 340	22 100
14	2 975	5 950	8 925	11 900	14 880	17 850	20.830	23 800
15	3 188	6 375	9.563	12 750	15 940	19 130	22 310	25 500
16	3 400	6 800	10 200	13 600	17 000	20 400	23.800	27.200
18	3 825	7 650	11 480	15 300	19.130	22 950	26 780	30.600
20	4.250	8 500	12 750	17 000	21 250	25 500	29 750	34 000
30	6 375	12 750	19 130	25 500	31 880	38 250	44.630	51 000
40	8 500	17.000	25 500	34 000	42 500	51 000	59.500	68 000
50	10.630	21 250	31 880	42 500	53 130	63 750	74 380	85.000
60	12 750	25 500	38 250	51 000	63 750	76 500	89.250	102.000
70	14.880	29 750	44 630	59.500	74.380	89 250	104 100	119 000
80	17 000	34 000	51 000	68 000	85 000	102 000	119 000	136.000
90	19 130	38 250	57.380	76 500	95 630	114.800	133.900	153 000
100	21 250	42 500	63 750	85 000	106 300	127 500	148.800	170 000

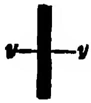
TABLE VII—(*Continued*)

WEIGHT OF STEEL PLATES, IN POUNDS PER LINEAR FOOT

Width Inches	Thickness, in Inches							
	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{1}{4}$	4781	5313	5844	6375	6906	7438	7969	85
$\frac{1}{2}$	9563	1 063	1 169	1 275	1 381	1 488	1 594	1 70
$\frac{3}{4}$	1 434	1 594	1 753	1 913	2 072	2 231	2 391	2 55
1	1 913	2 125	2 338	2 550	2 763	2 975	3 188	3 40
2	3 825	4 250	4 675	5 100	5 525	5 950	6 375	6 80
3	5 738	6 375	7 013	7 650	8 288	8 925	9 563	10 20
4	7 650	8 500	9 350	10 20	11 05	11 90	12 75	13 60
5	9 563	10 63	11 69	12 75	13 81	14 88	15 94	17 00
6	11 48	12 75	14 03	15 30	16 58	17 85	19 13	20 40
7	13 39	14 88	16 36	17 85	19 34	20 83	22 31	23 80
8	15 30	17 00	18 70	20 40	22 10	23 80	25 50	27 20
9	17 21	19 13	21 04	22 95	24 86	26 78	28 69	30 60
10	19 13	21 25	23 38	25 50	27 63	29 75	31 88	34 00
11	21 04	23 38	25 71	28 05	30 39	32 73	35 06	37 40
12	22 95	25 50	28 05	30 60	33 15	35 70	38 25	40 80
13	24 86	27 63	30 39	33 15	35 91	38 68	41 44	44 20
14	26 78	29 75	32 73	35 70	38 68	41 65	44 63	47 60
15	28 69	31 88	35 06	38 25	41 44	44 63	47 81	51 00
16	30 60	34 00	37 40	40 80	44 20	47 60	51 00	54 40
18	34 43	38 25	42 08	45 90	49 73	53 55	57 38	61 20
20	38 25	42 50	46 75	51 00	55 25	59 50	63 75	68 00
30	57 38	63 75	70 13	76 50	82 88	89 25	95 63	102 0
40	76 50	85 00	93 50	102 0	110 5	119 0	127 5	136 0
50	95 63	106 3	116 9	127 5	138 1	148 8	159 4	170 0
60	114 8	127 5	140 3	153 0	165 8	178 5	191 3	204 0
70	133 9	148 8	163 6	178 5	193 4	208 3	223 1	238 0
80	153 0	170 0	187 0	204 0	221 0	238 0	255 0	272 0
90	172 1	191 3	210 4	229 5	248 6	267 8	286 9	306 0
100	191 3	212 5	233 8	255 0	276 3	297 5	318 8	340 0

TABLE VIII

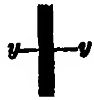
MOMENTS OF INERTIA OF RECTANGULAR SECTIONS



Width Inches	Thickness of Rectangle, in Inches						
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$
2	17	21	25	29	33	38	42
3	56	.70	84	98	1 13	1 27	1 41
4	1 33	1 67	2 00	2 33	2 67	3 00	3 33
5	2 60	3 26	3 91	4 56	5 21	5 86	6 51
6	4.50	5 63	6 75	7 88	9 00	10 13	11 25
7	7 15	8 93	10 72	12 51	14 29	16 08	17.86
8	10 67	13 33	16 00	18 67	21 33	24 00	26 67
9	15 19	18 98	22 78	26 58	30 38	34 17	37 97
10	20 83	26 04	31 25	36 46	41 67	46 87	52 08
11	27 73	34 66	41 59	48 53	55 46	62 39	69 32
12	36 00	45 00	54.00	63 00	72 00	81 00	90 00
13	45.77	57 21	68 66	80 10	91 54	102 98	114 43
14	57 17	71 46	85 75	100 04	114 33	128 63	142 92
15	70 31	87 89	105 47	123 05	140 63	158 20	175.78
16	85 33	106 67	128 00	149 33	170 67	192 00	213.33
17	102 35	127 94	153 53	179 12	204 71	230 30	255 89
18	121 50	151 88	182 25	212 63	243 00	273 38	303 75
19	142 90	178 62	214.34	250 07	285 79	321 52	357.24
20	166 67	208 33	250 00	291 67	333 33	375 00	416 67
22	221 83	277 29	332 75	388 21	443 67	499 13	554.58
24	288 00	360 00	432 00	504 00	576 00	648 00	720 00
26	366 17	457 71	549 25	640 79	732 33	823 88	915 42
28	457 33	571 67	686 00	800 33	914 67	1,029 00	1,143 33
30	562 50	703 13	843 75	984 38	1,125 00	1,265 63	1,406.25
32	682 67	853 33	1,024 00	1,194.67	1,365 33	1,536 00	1,706.67
34	818 83	1,023 54	1,228 25	1,432 96	1,637 67	1,842 38	2,047.08
36	972 00	1,215 00	1,458 00	1,701 00	1,944 00	2,187 00	2,430.00
38	1,143 17	1,428 96	1,714 75	2,000 54	2,286 33	2,572 13	2,857 92
40	1,333 33	1,666 67	2,000 00	2,333 33	2,666 67	3,000 00	3,333 33
42	1,543 50	1,929 38	2,315 25	2,701 13	3,087 00	3,472.88	3,858.75
44	1,774.67	2,218 33	2,662 00	3,105 67	3,549 33	3,993 00	4,436 67
46	2,027 83	2,534.79	3,041 75	3,548 71	4,055 67	4,562 63	5 069.58
48	2,304.00	2,880 00	3,456 00	4,032 00	4,608 00	5,184 00	5 760 00
52	2,929 33	3,661 67	4,394 00	5,126 33	5,858 67	6,591 00	7,323 33
56	3,658 67	4,573 33	5,488 00	6,402 67	7,317 33	8,232 00	9,146.67
60	4,500 00	5,625 00	6,750 00	7,875 00	9,000.00	10,125 00	11,250.00

TABLE VIII—(*Continued*)

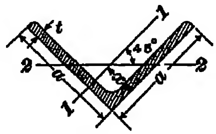
MOMENTS OF INERTIA OF RECTANGULAR SECTIONS



Width Inches	Thickness of Rectangle, in Inches					
	11/16	3/4	13/16	7/8	15/16	1
2	46	50	54	58	63	67
3	1 55	1 69	1 83	1 97	2 11	2 25
4	3 67	4 00	4 33	4 67	5 00	5 33
5	7 16	7 81	8 46	9 11	9 77	10 42
6	12 38	13 50	14 63	15 75	16 88	18 00
7	19 65	21 44	23 22	25 01	26 80	28 58
8	29 33	32 00	34 67	37 33	40 00	42 67
9	41 77	45 56	49 36	53 16	56 95	60 75
10	57 29	62 50	67 71	72 92	78 13	83 33
11	76 26	83 19	90 12	97 05	103 98	110 92
12	99 00	108 00	117 00	126 00	135 00	144 00
13	125 87	137 31	148 75	160 20	171 64	183 08
14	157 21	171 50	185 79	200 08	214 38	228 67
15	193 36	210 94	228 52	246 09	263 67	281 25
16	234 67	256 00	277 33	298 67	320 00	341 33
17	281 47	307 06	332 65	358 24	383 83	409 42
18	334.13	364 50	394 88	425 25	455 63	486 00
19	392 96	428 69	464 41	500 14	535 86	571 58
20	458 33	500 00	541 67	583 33	625 00	666 67
22	610 04	665 50	720 96	776 42	831 87	887 33
24	792 00	864 00	936 00	1,008 00	1,080 00	1,152 00
26	1,006 96	1,098 50	1,190 04	1,281 58	1,373 13	1,464 67
28	1,257 67	1,372 00	1,486 33	1,600 67	1,715 00	1,829 33
30	1,546 88	1,687 50	1,828 13	1,968 75	2,109 38	2,250 00
32	1,877 33	2,048 00	2,218 67	2,389 33	2,560 00	2,730 67
34	2,251 79	2,456 50	2,661 21	2,865 92	3,070 63	3,275 33
36	2,673 00	2,916 00	3,159 00	3,402 00	3,645 00	3,888 00
38	3,143 71	3,429 50	3,715 29	4,001 08	4,286 88	4,572 67
40	3,666 67	4,000 00	4,333 33	4,666 67	5,000 00	5,333 33
42	4,244 63	4,630 50	5,016 38	5,402 25	5,788 13	6,174 00
44	4,880 33	5,324 00	5,767 67	6,211 33	6,655 00	7,098 67
46	5,576 54	6,083 50	6,590 46	7,097 42	7,604 38	8,111 33
48	6,336 00	6,912 00	7,488 00	8,064 00	8,640 00	9,216 00
52	8,055 67	8,788 00	9,520 33	10,252 67	10,985 00	11,717 33
56	10,061 33	10,976 00	11,890 67	12,805 33	13,720 00	14,634.67
60	12,375 00	13,500 00	14,625 00	15,750 00	16,875 00	18,000.00

TABLE IX

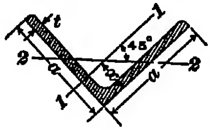
PROPERTIES OF STANDARD ANGLES HAVING EQUAL LEGS



1	2	3	4	5	6	7	8	9
Widths Inches	Thickness Inches	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Flange Inches	Moment of Inertia, Referred to the Inch Axis 1–1	Section Modulus, Referred to the Inch Axis 1–1	Radius of Gyration, Axis 1–1 Inches	Least Radius of Gyration, Axis 2–2 Inches
$a \times a$	t		A	x	I	Q	r	r'
1×1	1/8	80	23	30	022	031	30	19
1×1	3/16	1 16	34	32	030	044	30	19
1×1	1/4	1 49	44	34	037	056	29	19
1¼×1¼	1/8	1 02	30	36	044	049	38	.24
1¼×1¼	3/16	1 47	43	38	061	071	.38	24
1¼×1¼	1/4	1 91	56	40	077	091	37	24
1¼×1¼	5/16	2 32	68	42	0 090	11	36	24
1½×1½	3/16	1 79	53	44	11	10	46	.29
1½×1½	1/4	2 34	69	47	14	13	45	29
1½×1½	5/16	2 86	84	49	16	16	44	29
1½×1½	3/8	3 35	98	51	19	19	44	29
1¾×1¾	3/16	2 11	62	51	18	.14	54	34
1¾×1¾	1/4	2 76	81	53	23	19	53	34
1¾×1¾	5/16	3 39	1 00	55	27	23	52	34
1¾×1¾	3/8	3 98	1 17	57	31	26	51	.34
1¾×1¾	7/16	4 56	1 34	.59	35	30	51	34
2×2	3/16	2 43	71	57	27	19	62	.39
2×2	1/4	3 19	94	59	35	25	61	39
2×2	5/16	3 92	1 15	61	.42	.30	60	.39
2×2	3/8	4 62	1 36	64	48	.35	59	39
2×2	7/16	5 30	1 56	66	54	40	59	38
2½×2½	3/16	3 1	90	69	55	30	78	49
2½×2½	1/4	4 0	1 19	72	70	39	77	.49
2½×2½	5/16	5 0	1 46	74	85	48	.76	49
2½×2½	3/8	5 9	1 73	76	98	57	.75	.48
2½×2½	7/16	6 8	2 00	78	1 11	65	.75	.48
2½×2½	1/2	7 7	2 25	81	1.23	.72	74	.48

TABLE IX—(*Continued*)

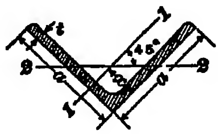
PROPERTIES OF STANDARD ANGLES HAVING EQUAL LEGS



1	2	3	4	5	6	7	8	9
Widths Inches $a \times a$	Thickness Inches t	Weight per Foot Pounds	Area of Section Square Inches A	Distance of Center of Gravity From Back of Flange Inches x	Moment of Inertia, Referred to the Inch Axis 1-1 I	Section Modulus, Referred to the Inch Axis 1-1 Q	Radius of Gyration, Axis 1-1 Inches r	Least Radius of Gyration, Axis 2-2 Inches r'
3×3	$\frac{1}{4}$	4 9	1 44	84	1 24	58	93	59
3×3	$\frac{5}{16}$	6 0	1 78	87	1 51	71	92	59
3×3	$\frac{3}{8}$	7 2	2 11	89	1 76	83	91	58
3×3	$\frac{7}{16}$	8 3	2 43	91	1 99	95	91	58
3×3	$\frac{1}{2}$	9 4	2 75	93	2 22	1 07	90	58
3×3	$\frac{9}{16}$	10 4	3 06	95	2 43	1 19	89	58
3×3	$\frac{5}{8}$	11 4	3 36	98	2 62	1 30	88	58
3½×3½	$\frac{5}{16}$	7 2	2 09	99	2 45	98	1 08	69
3½×3½	$\frac{3}{8}$	8 4	2 48	1 01	2 87	1 15	1 07	68
3½×3½	$\frac{7}{16}$	9 8	2 87	1 04	3 26	1 32	1 07	68
3½×3½	$\frac{1}{2}$	11 1	3 25	1 06	3 64	1 49	1 06	68
3½×3½	$\frac{9}{16}$	12 3	3 62	1 08	3 99	1 65	1 05	68
3½×3½	$\frac{5}{8}$	13 5	3 98	1 10	4 33	1 81	1 04	68
3½×3½	$\frac{11}{16}$	14 8	4 34	1 12	4 65	1 96	1 04	67
3½×3½	$\frac{3}{4}$	15 9	4 69	1 15	4 96	2 11	1 03	67
3½×3½	$\frac{13}{16}$	17 1	5 03	1.17	5 25	2 25	1 02	67
4×4	$\frac{5}{16}$	8 2	2 40	1 12	3 71	1 29	1 24	79
4×4	$\frac{3}{8}$	9 7	2 86	1 14	4 36	1 52	1 23	79
4×4	$\frac{7}{16}$	11 2	3 31	1 16	4 97	1 75	1 23	78
4×4	$\frac{1}{2}$	12 8	3 75	1 18	5 56	1 97	1 22	78
4×4	$\frac{9}{16}$	14 2	4 18	1 21	6 12	2 19	1 21	78
4×4	$\frac{5}{8}$	15 7	4 61	1 23	6 66	2 40	1 20	77
4×4	$\frac{11}{16}$	17 1	5 03	1 25	7 17	2 61	1 19	77
4×4	$\frac{3}{4}$	18 5	5 44	1 27	7 66	2 81	1 19	77
4×4	$\frac{13}{16}$	19 9	5 84	1 29	8 14	3 01	1 18	77
5×5	$\frac{3}{8}$	12 3	3 61	1 39	8 74	2.42	1 56	99
5×5	$\frac{7}{16}$	14 3	4 18	1.41	10 02	2.79	1.55	98
5×5	$\frac{1}{2}$	16 2	4 75	1 43	11 25	3 15	1.54	98

TABLE IX—(*Continued*)

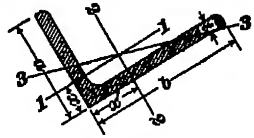
PROPERTIES OF STANDARD ANGLES HAVING EQUAL LEGS



1	2	3	4	5	6	7	8	9
Widths Inches $a \times a$	Thickness Inches t	Weight per Foot Pounds	Area of Section Square Inches A	Distance of Center of Gravity From Back of Flange Inches x	Moment of Inertia, Referred to the Inch Axis 1-1 I	Section Modulus, Referred to the Inch Axis 1-1 Q	Radius of Gyration, Axis 1-1 Inches r	Least Radius of Gyration, Axis 2-2 Inches r'
5×5	9/16	18.1	5.31	1.46	12.44	3.51	1.53	.98
5×5	5/8	20.0	5.86	1.48	13.58	3.86	1.52	.97
5×5	11/16	21.8	6.42	1.50	14.68	4.20	1.51	.97
5×5	3/4	23.6	6.94	1.52	15.74	4.53	1.51	.97
5×5	13/16	25.4	7.46	1.55	16.77	4.85	1.50	.97
5×5	7/8	27.2	7.99	1.57	17.75	5.17	1.49	.96
5×5	15/16	28.9	8.50	1.59	18.71	5.49	1.48	.96
5×5	1	30.6	9.00	1.61	19.64	5.80	1.48	.96
6×6	3/8	14.9	4.36	1.64	15.39	3.53	1.88	1.19
6×6	7/16	17.2	5.06	1.66	17.68	4.07	1.87	1.19
6×6	1/2	19.6	5.75	1.68	19.91	4.61	1.86	1.18
6×6	9/16	21.9	6.43	1.71	22.07	5.14	1.85	1.18
6×6	5/8	24.2	7.11	1.73	24.16	5.66	1.84	1.17
6×6	11/16	26.4	7.78	1.75	26.19	6.17	1.83	1.17
6×6	3/4	28.7	8.44	1.78	28.15	6.66	1.83	1.17
6×6	13/16	30.9	9.09	1.80	30.06	7.15	1.82	1.17
6×6	7/8	33.1	9.73	1.82	31.92	7.63	1.81	1.16
6×6	15/16	35.3	10.37	1.84	33.72	8.11	1.80	1.16
6×6	1	37.4	11.00	1.86	35.46	8.57	1.80	1.16
8×8	1/2	26.4	7.75	2.19	48.63	8.37	2.50	1.58
8×8	9/16	29.6	8.68	2.21	54.09	9.34	2.50	1.58
8×8	5/8	32.7	9.61	2.23	59.42	10.30	2.49	1.58
8×8	11/16	35.8	10.53	2.25	64.64	11.25	2.48	1.58
8×8	3/4	38.9	11.44	2.28	69.74	12.18	2.47	1.57
8×8	13/16	42.0	12.34	2.30	74.71	13.11	2.46	1.57
8×8	7/8	45.0	13.23	2.32	79.58	14.01	2.45	1.57
8×8	15/16	48.1	14.12	2.34	84.33	14.91	2.44	1.56
8×8	1	51.0	15.00	2.37	88.98	15.80	2.44	1.56
8×8	1 1/16	54.0	15.87	2.39	93.53	16.67	2.43	1.56
8×8	1 1/8	56.9	16.73	2.41	97.97	17.53	2.42	1.55

TABLE X

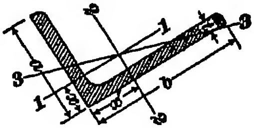
PROPERTIES OF STANDARD ANGLES HAVING UNEQUAL LEGS



1	2	3	4	5	6	7	8	9	10	11	12	13
Widths Inches	Thickness Inch	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Longer Flange Inches	Moment of Inertia Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Distance of Center of Gravity From Back of Shorter Flange Inches	Moment of Inertia Referred to the Inch Axis 2-2	Section Modulus Referred to the Inch Axis 2-2	Radius of Gyration, Axis 2-2 Inches	Least Radius of Gyration, Axis 3-3 Inch
$b \times a$	t		A	x	I	Q	r	x'	I'	Q'	r'	r''
2½×2	3/16	2 8	81	51	29	20	.60	76	51	29	79	43
2½×2	1/4	3 6	1 06	54	.37	25	59	.79	65	38	78	42
2½×2	5/16	4 5	1 31	56	45	31	58	81	79	47	78	42
2½×2	3/8	5 3	1 55	58	51	36	58	83	91	55	77	42
2½×2	7/16	6 0	1 78	60	58	41	57	85	1 03	62	76	42
2½×2	1/2	6 8	2 00	63	.64	46	56	88	1 14	70	75	42
3×2½	1/4	4 5	1 31	66	74	40	.75	91	1 17	56	95	53
3×2½	5/16	5 5	1 62	68	90	49	74	93	1 42	69	94	53
3×2½	3/8	6 5	1 92	.71	1 04	58	74	96	1 66	81	93	52
3×2½	7/16	7 5	2 21	73	1 18	.66	73	98	1 88	93	92	.52
3×2½	1/2	8 5	2 50	75	1 30	74	72	1 00	2 08	1 04	91	52
3×2½	9/16	9 4	2 78	77	1 42	82	72	1 02	2.28	1 15	91	52
3½×2½	1/4	4 9	1.44	61	78	41	74	1 11	1.80	75	1 12	54
3½×2½	5/16	6 0	1 78	64	94	.50	73	1 14	2 19	93	1 11	54
3½×2½	3/8	7 2	2 11	.66	1 09	.59	72	1.16	2 56	1 09	1 10	54
3½×2½	7/16	8 3	2 43	.68	1 23	68	71	1 18	2 91	1 26	1 09	54
3½×2½	1/2	9 4	2 75	70	1.36	76	70	1 20	3 24	1 41	1 09	53
3½×2½	9/16	10 4	3 06	.73	1 49	84	70	1 23	3 55	1 56	1 08	53
3½×2½	5/8	11 4	3 36	75	1 61	92	69	1 25	3 85	1 71	1 07	53
3½×2½	11/16	12 4	3 65	77	1 72	09	69	1 27	4 13	1 85	1 06	53
3½×3	5/16	6 6	1 93	81	1.58	72	90	1 06	2 33	95	1 10	63
3½×3	3/8	7 8	2 30	83	1 85	85	90	1 08	2 72	1 13	1 09	62
3½×3	7/16	9 0	2 65	85	2 09	98	89	1 10	3 10	1 29	1 08	.62
3½×3	1/2	10 2	3 00	.88	2 33	1 10	88	1 13	3 45	1 45	1 07	62
3½×3	9/16	11 4	3 34	90	2 55	1 21	.87	1 15	3 79	1 61	1 07	62
3½×3	5/8	12 5	3 67	92	2 76	1 33	87	1.17	4.11	1 76	1 06	62
3½×3	11/16	13.6	4 00	94	2 96	1 44	86	1 19	4 41	1 91	1 05	62
3½×3	3/4	14 7	4 31	.96	3 15	1 54	.85	1 21	4 70	2 05	1 04	62
3½×3	13/16	15.7	4.62	.98	3 33	1 65	85	1 23	4 98	2 20	1 04	.62
4×3	5/16	7 1	2 09	76	1.65	73	89	1 26	3.38	1.23	1.27	.65

TABLE X—(*Continued*)

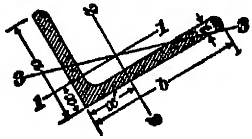
PROPERTIES OF STANDARD ANGLES HAVING UNEQUAL LEGS



1	2	3	4	5	6	7	8	9	10	11	12	13
Widths Inches	Thickness Inch	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Longer Flange Inches	Moment of Inertia Referred to the Inch Axis *1-1*	Section Modulus, Referred to the Inch Axis *1-1*	Radius of Gyration, Axis *1-1* Inches	Distance of Center of Gravity From Back of Shorter Flange Inches	Moment of Inertia, Referred to the Inch Axis *2-2*	Section Modulus Referred to the Inch Axis *2-2*	Radius of Gyration Axis *2-2* Inches	Least Radius of Gyration, Axis *3-3* Inch
$b \times a$	t		A	x	I	Q	r	x'	I'	Q'	r'	r''
4×3	$\frac{3}{8}$	8 4	2 48	.78	1 92	.87	88	1 28	3 96	1.46	1 26	64
4×3	$\frac{7}{16}$	9 8	2 87	80	2 18	.99	87	1 30	4 52	1 68	1 25	.64
4×3	$\frac{1}{2}$	11 1	3 25	83	2 42	1 12	86	1 33	5 05	1 89	1 25	64
4×3	$\frac{9}{16}$	12 3	3 62	85	2 66	1 23	86	1 35	5 55	2 09	1.24	64
4×3	$\frac{5}{8}$	13 6	3 98	87	2 87	1 35	.85	1 37	6 03	2 30	1 23	64
4×3	$\frac{11}{16}$	14 8	4 34	89	3 08	1 46	84	1 39	6 49	2 49	1 22	.64
4×3	$\frac{3}{4}$	15 9	4 69	92	3 28	1 57	84	1 42	6 93	2 68	1 22	64
4×3	$\frac{13}{16}$	17 1	5 03	94	3 47	1 68	83	1 44	7 35	2 87	1 21	64
5×3	$\frac{5}{16}$	8 2	2.40	68	1 75	75	85	1 68	6 26	1 89	1 61	66
5×3	$\frac{3}{8}$	9 7	2 86	.70	2 04	89	84	1 70	7 37	2.24	1 61	65
5×3	$\frac{7}{16}$	11 3	3 31	73	2 32	1 02	.84	1 73	8 43	2 58	1 60	65
5×3	$\frac{1}{2}$	12 8	3 75	.75	2 58	1.15	83	1 75	9 45	2 91	1 59	.65
5×3	$\frac{9}{16}$	14 2	4 18	77	2 83	1 27	82	1 77	10 43	3.23	1 58	65
5×3	$\frac{5}{8}$	15 7	4 61	80	3 06	1 39	.82	1 80	11 37	3 55	1 57	64
5×3	$\frac{11}{16}$	17 1	5 03	82	3 29	1 51	.81	1 82	12 28	3.86	1.56	.64
5×3	$\frac{3}{4}$	18 5	5 44	84	3 51	1 62	80	1 84	13 15	4 16	1.55	64
5×3	$\frac{13}{16}$	19 9	5 84	.86	3 71	1 74	.80	1 86	13 98	4 46	1 55	.64
5×3½	$\frac{5}{16}$	8 7	2 56	84	2 72	1 02	1 03	1 59	6 60	1 94	1 61	77
5×3½	$\frac{3}{8}$	10 4	3 05	86	3 18	1 21	1 02	1 61	7 78	2 29	1.60	.76
5×3½	$\frac{7}{16}$	12 0	3 53	.88	3 63	1 39	1 01	1 63	8 90	2 64	1 59	.76
5×3½	$\frac{1}{2}$	13 6	4 00	.91	4 05	1 56	1 01	1 66	9 99	2.99	1 58	75
5×3½	$\frac{9}{16}$	15 2	4 46	93	4 45	1 73	1 00	1 68	11 03	3 32	1.57	.75
5×3½	$\frac{5}{8}$	16 7	4 92	95	4 83	1 90	.99	1 70	12 03	3 65	1 56	.75
5×3½	$\frac{11}{16}$	18 3	5 37	97	5 20	2 06	98	1.72	12 99	3.97	1 56	75
5×3½	$\frac{3}{4}$	19 8	5 81	1 00	5.55	2.22	98	1 75	13 92	4 28	1.55	.75
5×3½	$\frac{13}{16}$	21 2	6 25	1 02	5 89	2 37	.97	1 77	14.81	4.58	1.54	.75
5×3½	$\frac{7}{8}$	22 7	6 67	1 04	6 21	2 52	96	1 79	15 67	4 88	1 53	.75
6×3½	$\frac{3}{8}$	11.6	3 42	.79	3 34	1 23	99	2 04	12.86	3 24	1.94	.77
6×3½	$\frac{7}{16}$	13.5	3 96	.81	3 81	1 41	.98	2 06	14 76	3.75	1 93	.76

TABLE X—(*Continued*)

PROPERTIES OF STANDARD ANGLES HAVING UNEQUAL LEGS



1	2	3	4	5	6	7	8	9	10	11	12	13
Widths Inches	Thickness Inch	Weight per Foot Pounds	Area of Section Square Inches	Distance of Center of Gravity From Back of Longer Flange Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration, Axis 1-1 Inches	Distance of Center of Gravity From Back of Shorter Flange Inches	Moment of Inertia, Referred to the Inch Axis 2-2	Section Modulus, Referred to the Inch Axis 2-2	Radius of Gyration, Axis 2-2 Inches	Least Radius of Gyration Axis 3-3 Inch
$b \times a$	t		A	x	I	Q	r	x'	I'	Q'	r'	r''
6×3½	½	15 3	4 50	83	4 25	1 59	97	2 08	16 59	4 24	1 92	76
6×3½	9/16	17 1	5 03	86	4 67	1 77	96	2 11	18 37	4 72	1 91	75
6×3½	⅝	18 9	5 55	88	5 08	1 94	96	2 13	20 08	5 19	1 90	75
6×3½	11/16	20 6	6 06	90	5 47	2 11	95	2 15	21 74	5 65	1 89	75
6×3½	¾	22 3	6 56	93	5 84	2 27	94	2 18	23 34	6 10	1 89	75
6×3½	13/16	24 0	7 06	.95	6 20	2 43	94	2 20	24 89	6 55	1 88	75
6×3½	⅞	25 7	7 55	.97	6 55	2 59	93	2 22	26 39	6 98	1 87	75
6×3½	15/16	27 3	8 03	99	6 88	2 74	93	2 24	27 84	7 41	1 86	74
6×3½	1	28 9	8 50	1 01	7 21	2 90	92	2 26	29 24	7 83	1 85	74
6×4	⅜	12 3	3 61	94	4 90	1 60	1 17	1 94	13 47	3 32	1 93	88
6×4	7/16	14 2	4 18	96	5 60	1 85	1 16	1 96	15 46	3 83	1 92	87
6×4	½	16 2	4 75	99	6 27	2 08	1 15	1 99	17 40	4 33	1 91	87
6×4	9/16	18 1	5 31	1 01	6 91	2 31	1.14	2 01	19 26	4 83	1 90	87
6×4	⅝	19 9	5.86	1 03	7 52	2 54	1.13	2 03	21 07	5 31	1 90	86
6×4	11/16	21 8	6 40	1 06	8 11	2.76	1.13	2 06	22 82	5 78	1 89	86
6×4	¾	23 6	6 94	1 08	8 68	2 97	1 12	2 08	24 51	6 25	1 88	86
6×4	13/16	25 4	7 46	1 10	9 23	3 18	1 11	2 10	26 15	6 70	1 87	.86
6×4	⅞	27 2	7 98	1 12	9 75	3 39	1 11	2 12	27 73	7 15	1 86	.86
6×4	15/16	28 9	8 50	1 14	10 26	3 59	1 10	2 14	29 26	7 59	1 86	.85
6×4	1	30 6	9.00	1 17	10 75	3 79	1 09	2 17	30 75	8 02	1 85	.85
7×3½	½	17 0	5 00	.78	4 41	1 62	.94	2.53	25 41	5 68	2 25	.89
7×3½	9/16	19 1	5 59	80	4 86	1 80	.93	2 55	28 18	6 33	2 25	.89
7×3½	⅝	21 0	6 17	82	5 28	1 97	93	2 57	30 86	6 97	2 24	89
7×3½	11/16	23 0	6 75	85	5 69	2 14	92	2 60	33 47	7 60	2 23	89
7×3½	¾	24 9	7 31	87	6 08	2 31	91	2 62	35 99	8 22	2 22	88
7×3½	13/16	26 8	7 87	.89	6 46	2 48	.91	2 64	38 45	8 82	2 21	.88
7×3½	⅞	28 7	8 42	.92	6 83	2 64	.90	2 67	40 82	9 42	2.20	88
7×3½	15/16	30 5	8 97	.94	7 18	2 80	.89	2 69	43 13	10 00	2 19	88
7×3½	1	32 3	9 50	96	7 53	2 96	.89	2 71	45 37	10.58	2.19	.88

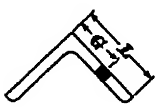
TABLE XI

THICKNESSES OF ANGLES FOR WHICH NOMINAL AND ACTUAL WIDTHS ARE EQUAL

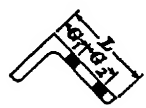
Equal Legs Inches	Thicknesses Inch	Unequal Legs Inches	Thicknesses Inch
8×8	½ and ¾	7×3½	½ and ¾
6×6	⅜ and 11/16	6×4	⅜ and 9/16
4×4	5/16 and ½	6×3½	⅜ and 9/16
3½×3½	5/16 and ½	5×3½	5/16 and ½
3×3	¼	5×3	5/16 and ½
2½×2½	3/16	4×3	5/16 and ½
2×2	3/16	3½×3	5/16
1¾×1¾	3/16	3½×2½	¼
1½×1½	⅛	3×2½	¼
1¼×1¼	⅛	2½×2	3/16

TABLE XII

STANDARD GAUGE LINES FOR ANGLES, IN INCHES

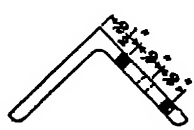


L	G	Maximum Rivet
8	4½	⅞
7	4	⅞
6	3½	⅞
5	3	⅞
4	2½	⅞
3½	2	⅞
3	1¾	⅞
2½	1⅜	¾
2	1⅛	⅝
1¾	1	½
1½	⅞	⅜
1¼	¾	⅜



L	G_1	G_2	Maximum Rivet
8	3	3	⅞
7	2½	3	⅞
6	2¼	2½	⅞
5	2	1¾	⅞

For 8-Inch Leg Only



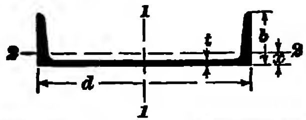
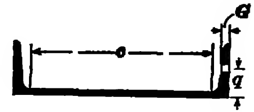
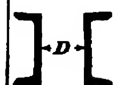


TABLE XIII

PROPERTIES OF STANDARD CHANNELS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Depth of Channel Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inch	Width of Flange Inches	Moment of Inertia, Referred to the Inch Axis *1–1*	Section Modulus, Referred to the Inch Axis *1–1*	Radius of Gyration, Axis *1–1* Inches	Moment of Inertia, Referred to the Inch Axis *2–2*	Section Modulus, Referred to the Inch Axis *2–2*	Radius of Gyration, Axis *2–2* Inches	Distance of Center of Gravity From Outside of Web Inches	Clear Distance Between Fillets Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inches	Maximum Flange Rivet or Bolt Inches	Distance Between Backs of L's to Make Radii of Gyration Equal
d		A	t	b	I	Q	r	I'	Q'	r'	x	c	q	G		D
3	4.00	1.19	.17	1.41	1.6	1.1	1.17	.20	.21	.41	.44	1¾	$\frac{15}{16}$	¼	⅜	
3	5 00	1 47	.26	1 50	1.8	1 2	1 12	25	.24	41	44	1¾	$\frac{15}{16}$	¼	⅜	
3	6 00	1 76	36	1 60	2 1	1 4	1 08	31	27	42	46	1¾	$\frac{15}{16}$	¼	⅜	
4	5.25	1.55	.18	1 58	3.8	1.9	1.56	.32	.29	.45	.46	2¾	1	$\frac{9}{32}$	½	2.06
4	6 25	1 84	25	1 65	4.2	2 1	1 51	38	32	45	.46	2¾	1	$\frac{9}{32}$	½	1 96
4	7 25	2 13	33	1 73	4 6	2 3	1 46	44	35	.46	46	2¾	1	$\frac{9}{32}$	½	1 85
5	6.50	1.95	.19	1 75	7.4	3.0	1.95	.48	.38	.50	.49	3¾	1	$\frac{5}{16}$	½	2.79
5	9 00	2 65	33	1 89	8 9	3 5	1 83	64	45	49	48	3¾	1¼	$\frac{5}{16}$	½	2 56
5	11 50	3 38	48	2 04	10 4	4.2	1 75	82	54	49	51	3¾	1¼	$\frac{5}{16}$	½	2 34

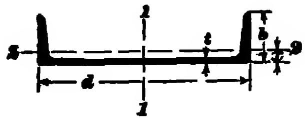
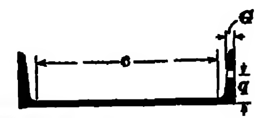


TABLE XIII—(*Continued*)

PROPERTIES OF STANDARD CHANNELS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Depth of Channel Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inch	Width of Flange Inches	Moment of Inertia, Referred to the Inch Axis 1–1	Section Modulus, Referred to the Inch Axis 1–1	Radius of Gyration Axis 1–1 Inches	Moment of Inertia, Referred to the Inch Axis 2–2	Section Modulus, Referred to the Inch Axis 2–2	Radius of Gyration, Axis 2–2 Inch	Distance of Center of Gravity From Outside of Web Inch	Clear Distance Between Fillets Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inch	Maximum Flange Rivet or Bolt Inch	Distance Between Backs of C's to Make Radii of Gyration Equal
d		*A*	*t*	*b*	*I*	*Q*	*r*	*I'*	*Q'*	*r'*	*x*	*c*	*q*	*G*		*D*
6	8.00	2 38	.20	1.92	13 0	4.3	2 34	.70	.50	.54	.52	4½	1⅛	11/32	⅝	3.52
6	10 50	3 09	32	2 04	15 1	5 0	2 21	88	57	53	50	4½	1⅛	11/32	⅝	3 28
6	13 00	3 82	44	2 16	17 3	5 8	2 13	1 07	65	53	52	4½	1⅜	11/32	⅝	3 09
6	15.50	4.56	56	2 28	19 5	6 5	2 07	1 28	74	53	55	4½	1⅜	11/32	⅝	2 91
7	9.75	2.85	.21	2.09	21.1	6.0	2.72	.98	63	.59	.55	5½	1½	11/32	⅝	4.22
7	12 25	3 60	32	2 20	24.2	6 9	2 59	1 19	71	57	53	5½	1½	11/32	⅝	3 99
7	14 75	4.34	.42	2 30	27 2	7 8	2 50	1 40	.79	57	53	5½	1½	11/32	⅝	3 80
7	17 25	5 07	53	2 41	30 2	8 6	2 44	1 62	87	56	55	5½	1½	⅜	⅝	3 64
7	19 75	5 81	.63	2 51	33 2	9 5	2 39	1 85	96	56	58	5½	1½	⅜	⅝	3 48

8	**11 25**	**3 35**	**.22**	**2.26**	**32.3**	**8.1**	**3.10**	**1.33**	**.79**	**.63**	**.58**	**6¼**	**1¼**	⅜	¾	**4 94**
8	13 75	4 04	31	2 35	36 0	9 0	2 98	1 55	87	62	56	6¼	1¼	⅜	¾	4.72
8	16 25	4 78	40	2 44	39 9	10 0	2 89	1 78	95	61	56	6¼	1½	⅜	¾	4.54
8	18 75	5 51	49	2 53	43 8	11 0	2 82	2 01	1 02	60	57	6¼	1½	⅜	¾	4.38
8	21 25	6 25	.58	2 62	47 8	11 9	2 76	2 25	1 11	60	59	6¼	1½	⅜	¾	4.23
9	**13 25**	**3 89**	**.23**	**2.43**	**47 3**	**10.5**	**3.49**	**1.77**	**.97**	**.67**	**.61**	**7¼**	**1⅜**	13/32	¾	**5.63**
9	15 00	4.41	29	2 49	50 9	11 3	3 40	1 95	1 03	66	59	7¼	1⅜	13/32	¾	5 49
9	20 00	5 88	45	2 65	60 8	13 5	3 21	2 45	1 19	65	58	7¼	1¾	⅜	¾	5 12
9	25 00	7 35	61	2 81	70 7	15 7	3 10	2 98	1 36	64	62	7¼	1¾	⅜	¾	4.84
10	**15 00**	**4.46**	**.24**	**2 60**	**66 9**	**13.4**	**3.87**	**2.30**	**1.17**	**72**	**.64**	**8¼**	**1½**	7/16	¾	**6 33**
10	20 00	5 88	38	2 74	78 7	15 7	3 66	2 85	1 34	70	61	8¼	1½	7/16	¾	5 97
10	25 00	7 35	53	2 89	91 0	18 2	3 52	3 40	1 50	68	62	8¼	2	⅜	¾	5 67
10	30 00	8 82	68	3 04	103 2	20 6	3 42	3 99	1 67	67	65	8¼	2	⅜	¾	5 40
10	35 00	10 29	82	3 18	115 5	23 1	3 35	4.66	1 87	67	69	8¼	2	⅜	¾	5 17
12	**20 50**	**6.03**	**.28**	**2.94**	**128 1**	**21.4**	**4.61**	**3.91**	**1.75**	**.81**	**.70**	**10**	**1¾**	15/32	¾	**7.67**
12	25 00	7 35	39	3 05	144.0	24.0	4 43	4.53	1 91	78	68	10	1¾	15/32	¾	7 36
12	30 00	8 82	51	3 17	161 6	26 9	4 28	5 21	2 09	77	68	10	2	15/32	¾	7 07
12	35 00	10 29	64	3 30	179 3	29 9	4.17	5 90	2 27	76	69	10	2	15/32	¾	6 81
12	40 00	11 76	76	3 42	196 9	32 8	4.09	6 63	2 46	75	72	10	2	15/32	¾	6 60
15	**33.00**	**9 90**	**40**	**3 40**	**312.6**	**41 7**	**5 62**	**8.23**	**3.16**	**.91**	**.79**	**12¼**	**1⅞**	21/32	¾	**9 50**
15	35 00	10 29	43	3 43	319 9	42 7	5 57	8 48	3 22	91	79	12¼	1⅞	21/32	¾	9 43
15	40 00	11 76	52	3 52	347 5	46 3	5 44	9 39	3 43	89	78	12¼	1⅞	21/32	¾	9 15
15	45 00	13 24	62	3 62	375 1	50 0	5 32	10 29	3 63	88	79	12¼	2¼	⅝	¾	8 92
15	50 00	14.71	72	3 72	402 7	53 7	5 23	11 22	3 85	87	80	12¼	2¼	⅝	¾	8 71
15	55 00	16 18	82	3 82	430 2	57 4	5 16	12 19	4 07	87	82	12¼	2¼	⅝	¾	8 53

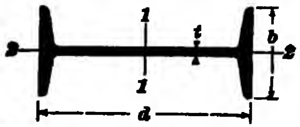
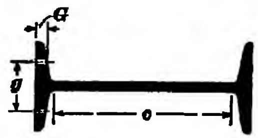
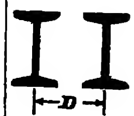


TABLE XIV

PROPERTIES OF STANDARD I BEAMS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Depth of Beam Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inches	Width of Flange Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration Axis 1-1 Inches	Moment of Inertia, Referred to the Inch Axis 2-2	Radius of Gyration Axis 2-2 Inches	Clear Distance Between Fillets Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inches	Maximum Flange Rivet or Bolt Inch	Distance Between Centers of Webs to Make Radii of Gyration Equal
d		A	t	b	I	Q	r	I'	r'	c	g	G		D
3	5.50	1.63	.17	2.33	2.5	1.7	1.23	.46	.53	$1\frac{3}{4}$	$1\frac{7}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	
3	6 50	1 91	26	2 42	2 7	1 8	1 19	53	52	$1\frac{3}{4}$	$1\frac{7}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	
3	7 50	2 21	36	2 52	2.9	1 9	1 15	60	52	$1\frac{3}{4}$	$1\frac{7}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	
4	7.50	2.21	.19	2.66	6.0	3.0	1.64	.77	.59	$2\frac{3}{4}$	$1\frac{1}{2}$	$\frac{9}{32}$	$\frac{1}{2}$	
4	8 50	2 50	26	2 73	6 4	3 2	1 59	.85	58	$2\frac{3}{4}$	$1\frac{1}{2}$	$\frac{9}{32}$	$\frac{1}{2}$	
4	9 50	2 79	34	2 81	6 7	3 4	1 55	93	.58	$2\frac{3}{4}$	$1\frac{1}{2}$	$\frac{9}{32}$	$\frac{1}{2}$	
4	10 50	3 09	41	2 88	7 1	3 6	1 52	1 01	.57	$2\frac{3}{4}$	$1\frac{1}{2}$	$\frac{9}{32}$	$\frac{1}{2}$	
5	9.75	2.87	.21	3 00	12.1	4.8	2.05	1.23	.65	$3\frac{1}{2}$	$1\frac{3}{4}$	$\frac{5}{16}$	$\frac{1}{2}$	
5	12.25	3 60	36	3 15	13 6	5 4	1 94	1 45	63	$3\frac{1}{2}$	$1\frac{3}{4}$	$\frac{5}{16}$	$\frac{1}{2}$	
5	14 75	4 34	50	3 29	15 2	6 1	1 37	1 70	63	$3\frac{1}{2}$	$1\frac{3}{4}$	$\frac{5}{16}$	$\frac{1}{2}$	
6	12.25	3.61	.23	3.33	21.8	7.3	2.46	1.85	.72	$4\frac{1}{2}$	2	$\frac{11}{32}$	$\frac{5}{8}$	4.70
6	14 75	4 34	.35	3 45	24 0	8 0	2.35	2 09	69	$4\frac{1}{2}$	2	$\frac{11}{32}$	$\frac{5}{8}$	4 49
6	17 25	5 07	47	3.57	26.2	8 7	2 27	2 36	68	$4\frac{1}{2}$	2	$\frac{11}{32}$	$\frac{5}{8}$	4 33
7	15.00	4.42	.25	3.66	36.2	10.4	2.86	2.67	.78	$5\frac{1}{4}$	$2\frac{1}{4}$	$\frac{3}{8}$	$\frac{5}{8}$	5.50
7	17.50	5 15	.35	3.76	39 2	11 2	2.76	2 94	76	$5\frac{1}{4}$	$2\frac{1}{4}$	$\frac{3}{8}$	$\frac{5}{8}$	5 31
7	20.00	5 88	46	3 87	42 2	12 1	2 68	3 24	74	$5\frac{1}{4}$	$2\frac{1}{4}$	$\frac{3}{8}$	$\frac{5}{8}$	5 15

8	**18.00**	**5.33**	**.27**	**4.00**	**56.9**	**14.2**	**3.27**	**3.78**	**.84**	**6¼**	**2¼**	**13/32**	**¾**	**6.32**
8	20 50	6 03	36	4 09	60 6	15 1	3 17	4 07	82	6¼	2¼	13/32	¾	6 12
8	23 00	6 76	45	4 18	64 5	16 1	3 09	4 39	81	6¼	2¼	13/32	¾	5 96
8	25 50	7 50	54	4 27	68 4	17 1	3 02	4 75	80	6¼	2¼	13/32	¾	5 82
9	**21.00**	**6.31**	**.29**	**4.33**	**84.9**	**18.9**	**3.67**	**5.16**	**.90**	**7**	**2½**	**7/16**	**¾**	**7.12**
9	25 00	7 35	41	4 45	91 9	20 4	3 54	5 65	88	7	2½	7/16	¾	6 86
9	30 00	8 82	57	4 61	101 9	22 6	3 40	6 42	85	7	2½	7/16	¾	6 58
9	35 00	10 29	73	4 77	111 8	24 8	3 29	7 31	84	7	2½	7/16	¾	6 36
10	**25 00**	**7.37**	**.31**	**4.66**	**122.1**	**24.4**	**4.07**	**6 89**	**.97**	**8**	**2⅝**	**15/32**	**¾**	**7.91**
10	30 00	8 82	45	4 80	134 2	26 8	3 90	7 65	93	8	2⅝	15/32	¾	7 57
10	35 00	10 29	60	4 95	146 4	29 3	3 77	8 52	91	8	2⅝	15/32	¾	7 32
10	40 00	11 76	75	5 10	158 7	31 7	3 67	9 50	90	8	2⅝	15/32	¾	7 12
12	**31.50**	**9.26**	**.35**	**5.00**	**215.8**	**36.0**	**4.83**	**9.50**	**1.01**	**9¾**	**2¾**	**17/32**	**¾**	**9 45**
12	35 00	10 29	44	5 09	228 3	38 0	4 71	10 07	99	9¾	2¾	17/32	¾	9 21
12	**40.00**	**11.84**	**46**	**5.25**	**268.9**	**44.8**	**4.77**	**13.81**	**1.08**	**9¼**	**3**	**21/32**	**¾**	**9.29**
12	45 00	13 24	58	5 37	285 7	47 6	4 65	14 89	1 06	9¼	3	21/32	¾	9 06
12	50 00	14 71	70	5 49	303 3	50 6	4 54	16 12	1 05	9¼	3	21/32	¾	8 83
12	55 00	16 18	.82	5 61	321 0	53 5	4 45	17 46	1.04	9¼	3	21/32	¾	8 65
15	**42.00**	**12.48**	**.41**	**5.50**	**441.7**	**58.9**	**5.95**	**14.62**	**1 08**	**12½**	**3**	**⅝**	**¾**	**11.70**
15	45 00	13 24	46	5 55	455 8	60 8	5 87	15 00	1 07	12½	3	⅝	¾	11 54
15	50.00	14 71	56	5 65	483 4	64 5	5 73	16 04	1 04	12½	3	⅝	¾	11 27
15	55 00	16 18	66	5 75	511 0	68 1	5 62	17 06	1 02	12½	3	⅝	¾	11 05
15	60 00	17 67	59	6 00	609 0	81 2	5 87	25 96	1 21	11¾	3¼	13/16	¾	11 45
15	65 00	19 12	69	6 10	636 0	84 8	5 77	27 42	1 20	11¾	3¼	13/16	¾	11 29
15	70 00	20 59	78	6 19	663 6	88 5	5 68	29 00	1 19	11¾	3¼	13/16	¾	11 11
15	75 00	22 06	88	6 29	691 2	92 2	5 60	30 68	1 18	11¾	3¼	13/16	¾	10 95
15	**80.00**	**23 81**	**.81**	**6.40**	**795.5**	**106.1**	**5.78**	**41 76**	**1.32**	**11**	**3¾**	**1 1/32**	**⅞**	**11.25**
15	85 00	25 00	89	6 48	817 8	109 0	5 72	43 57	1 32	11	3¾	1 1/32	⅞	11.13
15	90 00	26 47	99	6 58	845 4	112 7	5 65	45 91	1 32	11	3¾	1 1/32	⅞	10 99
15	95 00	27 94	1 08	6 67	872 9	116 4	5 59	48 37	1 32	11	3¾	1 1/32	⅞	10.86
15	100 00	29 41	1 18	6 77	900 5	120 1	5 53	50 98	1 31	11	3¾	1 1/32	⅞	10 75
18	**55.00**	**15.93**	**.46**	**6.00**	**795.6**	**88.4**	**7.07**	**21.19**	**1.15**	**15¼**	**3¼**	**11/16**	**⅞**	**13.95**
18	60.00	17 65	.56	6 10	841 8	93 5	6 91	22 38	1 13	15¼	3¼	11/16	⅞	13 63
18	65 00	19 12	.64	6 18	881 5	97 9	6 79	23 47	1 11	15¼	3¼	11/16	⅞	13 40
18	70.00	20 59	.72	6 26	921 3	102 4	6 69	24 62	1 09	15¼	3¼	11/16	⅞	13 20

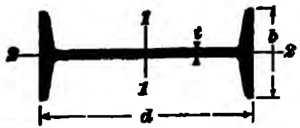
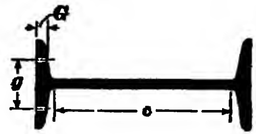


TABLE XIV—(*Continued*)

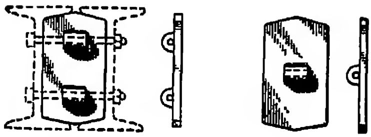
PROPERTIES OF STANDARD I BEAMS



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Depth of Beam Inches	Weight per Foot Pounds	Area of Section Square Inches	Thickness of Web Inches	Width of Flange Inches	Moment of Inertia, Referred to the Inch Axis 1-1	Section Modulus, Referred to the Inch Axis 1-1	Radius of Gyration Axis 1-1 Inches	Moment of Inertia, Referred to the Inch Axis 2-2	Radius of Gyration Axis 2-2 Inches	Clear Distance Between Fillets Inches	Gauge of Flanges Inches	Grip of Flange Rivets Inches	Maximum Flange Rivet or Bolt Inch	Distance Between Centers of Webs to Make Radii of Gyration Equal
d		*A*	*t*	*b*	*I*	*Q*	*r*	*I'*	*r'*	*c*	*g*	*G*		*D*
20	**65.00**	**19.08**	**.50**	**6.25**	**1,169.6**	**117.0**	**7.83**	**27.86**	**1.21**	**17**	**3½**	25/32	7/8	**15.47**
20	70 00	20.59	.58	6 33	1,219 9	122 0	7 70	29 04	1 19	17	3½	25/32	7/8	15 21
20	75 00	22 06	65	6 40	1,268 9	126 9	7 58	30 25	1 17	17	3½	25/32	7/8	14 98
20	**80.00**	**23.73**	**.60**	**7.00**	**1,466.5**	**146.7**	**7.86**	**45.81**	**1.39**	**16½**	**4**	29/32	7/8	**15.47**
20	85 00	25 00	66	7 06	1,508 7	150 9	7 77	47 25	1 37	16½	4	29/32	7/8	15 30
20	90 00	26 47	.74	7 14	1,557.8	155 8	7 67	48 98	1 36	16½	4	29/32	7/8	15 10
20	95 00	27 94	81	7 21	1,606 8	160 7	7 58	50 78	1 35	16½	4	29/32	7/8	14 92
20	100 00	29 41	88	7 28	1,655 8	165 6	7 50	52 65	1 34	16½	4	29/32	7/8	14 76
24	**80.00**	**23.32**	**.50**	**7.00**	**2,087.9**	**174.0**	**9.46**	**42.86**	**1.36**	**20¾**	**4**	27/32	7/8	**18.72**
24	85 00	25 00	57	7 07	2,168 6	180 7	9 31	44 35	1 33	20¾	4	27/32	7/8	18 43
24	90 00	26 47	63	7 13	2,239 1	186 6	9 20	45 70	1 31	20¾	4	27/32	7/8	18 21
24	95 00	27 94	69	7 19	2,309 6	192 5	9 09	47 10	1 30	20¾	4	27/32	7/8	17 99
24	100 00	29 41	75	7 25	2,380 3	198 4	9 00	48.56	1 28	20¾	4	27/32	7/8	17 82

TABLE XV

CAST-IRON SEPARATORS



Depth of Beam Inches	Weight of Beam Pounds per Foot	Center to Center of Beams Inches	Center to Center of Bolts Inches	Thickness of Separator Inch	Weight of Separator Pounds
6	12 25	4	*	½	4
7	15 00	4¼	*	½	4
8	18 00	4½	*	½	6
9	21 00	5	*	½	7
10	25 00	5½	*	½	8
12	31 50	5¾	*	½	10
12	40 00	6	*	½	10
12	31 50	5¾	5	½	11
12	40 00	6	5	½	11
15	42 00	6¼	7½	½	15
15	60 00	6¾	7½	½	15
15	80 00	7¼	7½	½	15
18	55 00	6¾	9	⅝	16
20	65 00	7	12	⅝	25
20	80 00	7¾	12	⅝	28
24	80 00	7¾	12	⅝	32

* For these beams, but one bolt is used.

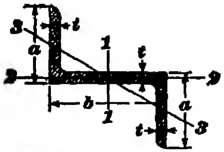
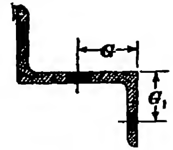


TABLE XVI

PROPERTIES OF Z BARS



1	2	3	4	5	6	7	8	9	10	11	12	13	14
Depth of Bar Inches	Length of Legs Inches	Thickness of Web and Legs Inch	Weight per Foot Pounds	Area of Section Square Inches	Moment of Inertia, Referred to the I ch Axis *1–1*	Section Modulus, Referred to the Inch Axis *1–1*	Radius of Gyration, Axis *1–1* Inches	Moment of Inertia, Referred to the Inch Axis *2–2*	Section Modulus, Referred to the Inch Axis *2–2*	Radius of Gyration, Axis *2–2* Inches	Least Radius of Gyration, Axis *3–3* Inch	Gauge in Web Inches	Gauge in Leg Inches
b	a	t		A	I	Q	r	I'	Q'	r'	r''	G	G_1
3	$2\frac{11}{16}$	$\frac{1}{4}$	6 7	1 97	2 87	1 92	1 21	2 81	1 10	1 19	55	$1\frac{1}{2}$	$1\frac{5}{8}$
$3\frac{1}{16}$	$2\frac{3}{4}$	$\frac{5}{16}$	8 4	2 48	3 64	2 38	1 21	3 64	1 40	1 21	55	$1\frac{1}{2}$	$1\frac{5}{8}$
3	$2\frac{11}{16}$	$\frac{3}{8}$	9 7	2 86	3.85	2 57	1 16	3 92	1 57	1 17	54	$1\frac{1}{2}$	$1\frac{5}{8}$
$3\frac{1}{16}$	$2\frac{3}{4}$	$\frac{7}{16}$	11 4	3 36	4 57	2 98	1 17	4 75	1 88	1 19	55	$1\frac{1}{2}$	$1\frac{5}{8}$
3	$2\frac{11}{16}$	$\frac{1}{2}$	12 5	3.69	4 59	3 06	1 12	4 85	1 99	1 15	53	$1\frac{1}{2}$	$1\frac{5}{8}$
$3\frac{1}{16}$	$2\frac{3}{4}$	$\frac{9}{16}$	14 2	4 18	5 26	3 43	1 12	5 68	2 30	1.17	54	$1\frac{1}{2}$	$1\frac{5}{8}$
4	$3\frac{1}{16}$	$\frac{1}{4}$	8 2	2 41	6 28	3.14	1 62	4 23	1 44	1 33	.67	2	2
$4\frac{1}{16}$	$3\frac{1}{8}$	$\frac{5}{16}$	10 3	3 03	7 94	3 91	1 62	5 46	1 84	1 34	.68	2	2
$4\frac{1}{8}$	$3\frac{3}{16}$	$\frac{3}{8}$	12 4	3 66	9 63	4 67	1 62	6 77	2 26	1 36	.69	2	2

4	$3\frac{1}{16}$	$\frac{7}{16}$	13 8	4 05	9.66	4 83	1 54	6 73	2 37	1 29	66	2	2
$4\frac{1}{16}$	$3\frac{1}{8}$	$\frac{1}{2}$	15 8	4.66	11.18	5 50	1 55	7 96	2 77	1 31	67	2	2
$4\frac{1}{8}$	$3\frac{3}{16}$	$\frac{9}{16}$	17 9	5 27	12 74	6 18	1.55	9 26	3 19	1 32	68	2	2
4	$3\frac{1}{16}$	$\frac{5}{8}$	18 9	5 55	12 11	6 05	1 48	8 73	3 18	1 25	65	2	2
$4\frac{1}{16}$	$3\frac{1}{8}$	$\frac{11}{16}$	20 9	6.14	13 52	6 65	1 48	9 95	3 58	1 27	67	2	2
$4\frac{1}{8}$	$3\frac{3}{16}$	$\frac{3}{4}$	23 0	6 75	14 97	7 26	1 49	11 24	4 00	1 29	68	2	2
5	$3\frac{1}{4}$	$\frac{5}{16}$	11 6	3 40	13 36	5 34	1 98	6 18	2 00	1 35	75	$2\frac{1}{2}$	$2\frac{1}{8}$
$5\frac{1}{16}$	$3\frac{5}{16}$	$\frac{3}{8}$	13 9	4 10	16 18	6 39	1 99	7 65	2 45	1 37	76	$2\frac{1}{2}$	$2\frac{1}{8}$
$5\frac{1}{8}$	$3\frac{3}{8}$	$\frac{7}{16}$	16 4	4 81	19 07	7 44	1 99	9 20	2 92	1 38	76	$2\frac{1}{2}$	$2\frac{1}{8}$
5	$3\frac{1}{4}$	$\frac{1}{2}$	17 9	5 25	19 19	7 68	1 91	9 05	3 02	1.31	74	$2\frac{1}{2}$	$2\frac{1}{8}$
$5\frac{1}{16}$	$3\frac{5}{16}$	$\frac{9}{16}$	20 2	5 94	21 83	8 62	1 92	10 51	3 47	1 33	75	$2\frac{1}{2}$	$2\frac{1}{8}$
$5\frac{1}{8}$	$3\frac{3}{8}$	$\frac{5}{8}$	22 6	6 64	24 53	9 57	1 92	12 06	3 94	1 35	76	$2\frac{1}{2}$	$2\frac{1}{8}$
5	$3\frac{1}{4}$	$\frac{11}{16}$	23 7	6 96	23.68	9 47	1 84	11 37	3 91	1 28	73	$2\frac{1}{2}$	$2\frac{1}{8}$
$5\frac{1}{16}$	$3\frac{5}{16}$	$\frac{3}{4}$	26 0	7 64	26 16	10 34	1 85	12 83	4 37	1 30	74	$2\frac{1}{2}$	$2\frac{1}{8}$
$5\frac{1}{8}$	$3\frac{3}{8}$	$\frac{13}{16}$	28 3	8 33	28 70	11 20	1 86	14.37	4 84	1 31	76	$2\frac{1}{2}$	$2\frac{1}{8}$
6	$3\frac{1}{2}$	$\frac{3}{8}$	15 6	4 59	25 32	8 44	2 35	9 11	2 75	1 41	.83	3	$2\frac{1}{4}$
$6\frac{1}{16}$	$3\frac{9}{16}$	$\frac{7}{16}$	18 3	5 39	29 80	9 83	2 35	10 94	3 27	1 43	83	3	$2\frac{1}{4}$
$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{1}{2}$	21 0	6 19	34 36	11 22	2 36	12 87	3 81	1 44	84	3	$2\frac{1}{4}$
6	$3\frac{1}{2}$	$\frac{9}{16}$	22 7	6 68	34 64	11 55	2 28	12 59	3 91	1 37	81	3	$2\frac{1}{4}$
$6\frac{1}{16}$	$3\frac{9}{16}$	$\frac{5}{8}$	25 4	7 46	38 87	12 82	2 28	14 41	4 44	1 39	82	3	$2\frac{1}{4}$
$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{11}{16}$	28 1	8 25	43 18	14 10	2 29	16 34	4 98	1 41	84	3	$2\frac{1}{4}$
6	$3\frac{1}{2}$	$\frac{3}{4}$	29 3	8 63	42 12	14 04	2 21	15 44	4 94	1 34	81	3	$2\frac{1}{4}$
$6\frac{1}{16}$	$3\frac{9}{16}$	$\frac{13}{16}$	31 9	9 39	46 13	15 22	2 22	17 27	5 47	1 36	82	3	$2\frac{1}{4}$
$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{7}{8}$	34 6	10 17	50 22	16 40	2 22	19 18	6 02	1 37	83	3	$2\frac{1}{4}$

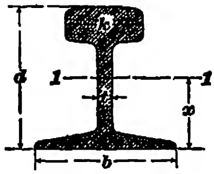
TABLE XVII
DECIMAL EQUIVALENTS

Fractional Parts of an Inch	Inches 0	1	2	3	4	5	6	7	8	9	10	11	Decimal Equivalents of Fractional Parts of an Inch	
	Decimal Parts of a Foot													
	0000	0833	1667	2500	3333	4167	5000	5833	6667	7500	8333	9167		
$\frac{1}{32}$	0026	0859	1693	2526	3359	4193	5026	5859	6693	7526	8359	9193	$\frac{1}{32}$	0313
$\frac{1}{16}$	0052	0885	1719	2552	3385	4219	5052	5885	6719	7552	8385	9219	$\frac{1}{16}$	0625
$\frac{3}{32}$	0078	0911	1745	2578	3411	4245	5078	5911	6745	7578	8411	9245	$\frac{3}{32}$	0938
$\frac{1}{8}$	0104	0938	1771	2604	3438	4271	5104	5938	6771	7604	8438	9271	$\frac{1}{8}$	1250
$\frac{5}{32}$	0130	0964	1797	2630	3464	4297	5130	5964	6797	7630	8464	9297	$\frac{5}{32}$	1563
$\frac{3}{16}$	0156	0990	1823	2656	3490	4323	5156	5990	6823	7656	8490	9323	$\frac{3}{16}$	1875
$\frac{7}{32}$	0182	1016	1849	2682	3516	4349	5182	6016	6849	7682	8516	9349	$\frac{7}{32}$	2188
$\frac{1}{4}$	0208	1042	.1875	2708	3542	4375	5208	6042	6875	7708	8542	9375	$\frac{1}{4}$	2500
$\frac{9}{32}$	0234	1068	1901	2734	3568	4401	5234	6068	6901	7734	8568	9401	$\frac{9}{32}$	2813
$\frac{5}{16}$	0260	.1094	1927	2760	3594	.4427	5260	6094	6927	.7760	8594	9427	$\frac{5}{16}$	3125
$\frac{11}{32}$	0286	1120	1953	2786	3620	4453	5286	6120	6953	7786	8620	9453	$\frac{11}{32}$	3438

$\frac{3}{8}$	.0313	1146	1979	2813	.3646	4479	5313	6146	6979	7813	8646	9479	$\frac{3}{8}$	3750
$\frac{13}{32}$	0339	1172	2005	2839	3672	4505	5339	6172	7005	7839	8672	9505	$\frac{13}{32}$	4063
$\frac{7}{16}$	0365	1198	2031	2865	3698	4531	5365	6198	7031	7865	8698	9531	$\frac{7}{16}$	4375
$\frac{15}{32}$	0391	1224	2057	2891	.3724	4557	5391	6224	7057	7891	8724	9557	$\frac{15}{32}$	4688
$\frac{1}{2}$	.0417	1250	2083	2917	3750	4583	5417	6250	7083	7917	.8750	9583	$\frac{1}{2}$	5000
$\frac{17}{32}$	0443	.1276	2109	2943	3776	4609	5443	6276	7109	7943	8776	9609	$\frac{17}{32}$	5313
$\frac{9}{16}$	0469	1302	2135	2969	.3802	4635	5469	6302	7135	7969	8802	9635	$\frac{9}{16}$	5625
$\frac{19}{32}$	0495	.1328	2161	2995	.3828	4661	5495	6328	7161	7995	8828	9661	$\frac{19}{32}$	5938
$\frac{5}{8}$	0521	1354	2188	3021	.3854	4688	5521	6354	7188	8021	8854	9688	$\frac{5}{8}$	6250
$\frac{21}{32}$	0547	1380	2214	3047	.3880	4714	5547	6380	7214	8047	8880	9714	$\frac{21}{32}$	6563
$\frac{11}{16}$	0573	1406	2240	3073	.3906	4740	5573	6406	7240	8073	8906	9740	$\frac{11}{16}$	6875
$\frac{23}{32}$	0599	1432	2266	3099	3932	4766	5599	6432	7266	8099	8932	9766	$\frac{23}{32}$	7188
$\frac{3}{4}$	0625	1458	2292	3125	3958	4792	5625	6458	7292	8125	8958	9792	$\frac{3}{4}$	7500
$\frac{25}{32}$	0651	1484	2318	3151	3984	4818	5651	6484	7318	8151	8984	9818	$\frac{25}{32}$	7813
$\frac{13}{16}$	0677	1510	.2344	3177	4010	4844	5677	6510	.7344	8177	9010	9844	$\frac{13}{16}$	8125
$\frac{27}{32}$	0703	1536	2370	3203	4036	4870	5703	6536	7370	8203	9036	9870	$\frac{27}{32}$	8438
$\frac{7}{8}$	0729	1563	2396	3229	4063	4896	5729	6563	7396	8229	9063	9896	$\frac{7}{8}$	8750
$\frac{29}{32}$	0755	1589	2422	3255	4089	4922	5755	6589	7422	8255	9089	9922	$\frac{29}{32}$	9063
$\frac{15}{16}$	0781	1615	2448	3281	4115	4948	5781	6615	7448	8281	9115	9948	$\frac{15}{16}$	9375
$\frac{31}{32}$	0807	1641	2474	3307	4141	4974	5807	6641	7474	8307	9141	9974	$\frac{31}{32}$	9688

TABLE XVIII

PROPERTIES AND PRINCIPAL DIMENSIONS OF STANDARD T RAILS



1	2	3	4	5	6	7	8	9
							Axis 1-1	
Weight per Yard Pounds	Area Square Inches	b Inches	d Inches	k Inches	t Inch	x Inches	Moment of Inertia I	Section Modulus Q
16	1 57	$2\frac{1}{4}$	$2\frac{1}{4}$	$1\frac{3}{16}$	$\frac{7}{32}$	1 10	1 13	99
20	2 00	$2\frac{3}{8}$	$2\frac{3}{8}$	$1\frac{3}{8}$	$\frac{1}{4}$	1 2	1 5	1 2
25	2 5	$2\frac{3}{4}$	$2\frac{3}{4}$	$1\frac{1}{2}$	$\frac{19}{64}$	1 4	2 4	1.7
30	2 9	$3\frac{5}{16}$	3	$1\frac{11}{16}$	$\frac{11}{32}$	1 5	3 7	2 4
35	3 4	$3\frac{3}{8}$	$3\frac{1}{16}$	$1\frac{27}{32}$	$\frac{13}{32}$	1 5	4 3	2 8
40	3 9	$3\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{27}{32}$	$\frac{23}{64}$	1 7	6 0	3.4
45	4 4	$3\frac{3}{4}$	$3\frac{3}{4}$	$1\frac{59}{64}$	$\frac{3}{8}$	1.8	7 6	3 9
50	4 9	$3\frac{7}{8}$	$3\frac{7}{8}$	$2\frac{1}{8}$	$\frac{7}{16}$	1 9	10 1	5.1
55	5 4	$4\frac{1}{16}$	$4\frac{1}{16}$	$2\frac{1}{4}$	$\frac{15}{32}$	2 0	12 2	5 9
60	5 9	$4\frac{1}{4}$	$4\frac{1}{4}$	$2\frac{3}{8}$	$\frac{31}{64}$	2.1	14 7	6 7
65	6 4	$4\frac{7}{16}$	$4\frac{7}{16}$	$2\frac{13}{32}$	$\frac{1}{2}$	2 1	17 0	7 4
70	6 9	$4\frac{5}{8}$	$4\frac{5}{8}$	$2\frac{7}{16}$	$\frac{33}{64}$	2 2	20 0	8.4
75	7.4	$4\frac{13}{16}$	$4\frac{13}{16}$	$2\frac{15}{32}$	$\frac{17}{32}$	2 3	23 0	9 1
80	7 8	5	5	$2\frac{1}{2}$	$\frac{35}{64}$	2 4	26 7	10 1
85	8.3	$5\frac{3}{16}$	$5\frac{3}{16}$	$2\frac{9}{16}$	$\frac{9}{16}$	2 5	30 5	11.2
90	8.8	$5\frac{3}{8}$	$5\frac{3}{8}$	$2\frac{5}{8}$	$\frac{9}{16}$	2 6	35.2	12 6
100	9.8	$5\frac{3}{4}$	$5\frac{3}{4}$	$2\frac{3}{4}$	$\frac{9}{16}$	2 8	44.4	15.0

TABLE XIX

SIZES OF FULL AND COUNTERSUNK RIVET HEADS

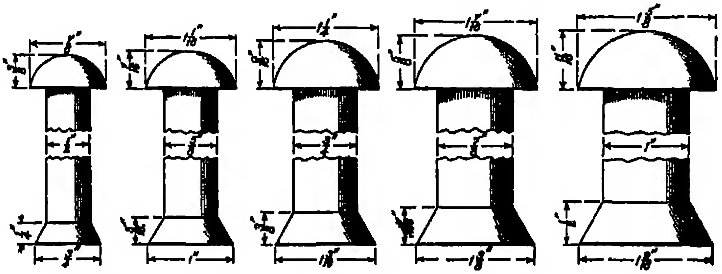


TABLE XX

ADDITIONAL LENGTH OF RIVET TO FORM ONE HEAD

Grip of Rivet Inches	Button Heads					Countersunk Rivets				
	Diameter of Rivet Inches					Diameter of Rivet Inches				
	½	⅝	¾	⅞	1	½	⅝	¾	⅞	1
½ to 1¼	1	1¼	1⅜	1½	1⅝	½	¾	¾	¾	¾
1⅜ to 3	1⅛	1⅜	1½	1⅝	1¾	⅝	⅞	⅞	⅞	⅞
3⅛ to 4⅝	1⅛	1½	1⅝	1¾	1⅞	⅝	1	1	1	1
4¾ to 6	1¼	1⅝	1¾	1⅞	2	¾	1⅛	1⅛	1⅛	1⅛

TABLE XXI

WEIGHT OF 100 RIVET HEADS

Diameter of Rivet, Inches	½	⅝	¾	⅞	1
Weight of 100 Heads Pounds	5 5	8 5	12.3	16.7	21.8

TABLE XXII

MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	1½	1⅝	1¾	1⅞	2	2⅛	2¼
	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″
2	0-3	0-3¼	0-3½	0-3¾	0-4	0-4¼	0-4½
3	0-4½	0-4⅞	0-5¼	0-5⅝	0-6	0-6⅜	0-6¾
4	0-6	0-6½	0-7	0-7½	0-8	0-8½	0-9
5	0-7½	0-8⅛	0-8¾	0-9⅜	0-10	0-10⅝	0-11¼
6	0-9	0-9¾	0-10½	0-11¼	1-0	1-¾	1-1½
7	0-10½	0-11⅜	1-¼	1-1⅛	1-2	1-2⅞	1-3¾
8	1-0	1-1	1-2	1-3	1-4	1-5	1-6
9	1-1½	1-2⅝	1-3¾	1-4⅞	1-6	1-7⅛	1-8¼
10	1-3	1-4¼	1-5½	1-6¾	1-8	1-9¼	1-10½
11	1-4½	1-5⅞	1-7¼	1-8⅝	1-10	1-11⅜	2-¾
12	1-6	1-7½	1-9	1-10½	2-0	2-1½	2-3
13	1-7½	1-9⅛	1-10¾	2-⅜	2-2	2-3⅝	2-5¼
14	1-9	1-10¾	2-½	2-2¼	2-4	2-5¾	2-7½
15	1-10½	2-⅜	2-2¼	2-4⅛	2-6	2-7⅞	2-9¾
16	2-0	2-2	2-4	2-6	2-8	2-10	3-0
17	2-1½	2-3⅝	2-5¾	2-7⅞	2-10	3-⅛	3-2¼
18	2-3	2-5¼	2-7½	2-9¾	3-0	3-2¼	3-4½
19	2-4½	2-6⅞	2-9¼	2-11⅝	3-2	3-4⅜	3-6¾
20	2-6	2-8½	2-11	3-1½	3-4	3-6½	3-9
21	2-7½	2-10⅛	3-¾	3-3⅜	3-6	3-8⅝	3-11¼
22	2-9	2-11¾	3-2½	3-5¼	3-8	3-10¾	4-1½
23	2-10½	3-1⅜	3-4¼	3-7⅛	3-10	4-⅞	4-3¾
24	3-0	3-3	3-6	3-9	4-0	4-3	4-6
25	3-1½	3-4⅝	3-7¾	3-10⅞	4-2	4-5⅛	4-8¼
26	3-3	3-6¼	3-9½	4-¾	4-4	4-7¼	4-10½
27	3-4½	3-7⅞	3-11¼	4-2⅝	4-6	4-9⅜	5-¾
28	3-6	3-9½	4-1	4-4½	4-8	4-11½	5-3
29	3-7½	3-11⅛	4-2¾	4-6⅜	4-10	5-1⅝	5-5¼
30	3-9	4-¾	4-4½	4-8¼	5-0	5-3¾	5-7½
31	3-10½	4-2⅜	4-6¼	4-10⅛	5-2	5-5⅞	5-9¾
32	4-0	4-4	4-8	5-0	5-4	5-8	6-0

TABLE XXII—(*Continued*)

MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	2⅜	2½	2⅝	2¾	2⅞	3	3⅛
	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″
2	0–4¾	0–5	0–5¼	0–5½	0–5¾	0–6	0–6¼
3	0–7⅛	0–7½	0–7⅞	0–8¼	0–8⅝	0–9	0–9⅜
4	0–9½	0–10	0–10½	0–11	0–11½	1–0	1–½
5	0–11⅞	1–½	1–1⅛	1–1¾	1–2⅜	1–3	1–3⅝
6	1–2¼	1–3	1–3¾	1–4½	1–5¼	1–6	1–6¾
7	1–4⅝	1–5½	1–6⅜	1–7¼	1–8⅛	1–9	1–9⅞
8	1–7	1–8	1–9	1–10	1–11	2–0	2–1
9	1–9⅜	1–10½	1–11⅝	2–¾	2–1⅞	2–3	2–4⅛
10	1–11¾	2–1	2–2¼	2–3½	2–4¾	2–6	2–7¼
11	2–2⅛	2–3½	2–4⅞	2–6¼	2–7⅝	2–9	2–10⅜
12	2–4½	2–6	2–7½	2–9	2–10½	3–0	3–1½
13	2–6⅞	2–8½	2–10⅛	2–11¾	3–1⅜	3–3	3–4⅝
14	2–9¼	2–11	3–¾	3–2½	3–4¼	3–6	3–7¾
15	2–11⅝	3–1½	3–3⅜	3–5¼	3–7⅛	3–9	3–10⅞
16	3–2	3–4	3–6	3–8	3–10	4–0	4–2
17	3–4⅜	3–6½	3–8⅝	3–10¾	4–⅞	4–3	4–5⅛
18	3–6¾	3–9	3–11¼	4–1½	4–3¾	4–6	4–8¼
19	3–9⅛	3–11½	4–1⅞	4–4¼	4–6⅝	4–9	4–11⅜
20	3–11½	4–2	4–4½	4–7	4–9½	5–0	5–2½
21	4–1⅞	4–4½	4–7⅛	4–9¾	5–⅜	5–3	5–5⅝
22	4–4¼	4–7	4–9¾	5–½	5–3¼	5–6	5–8¾
23	4–6⅝	4–9½	5–⅜	5–3¼	5–6⅛	5–9	5–11⅞
24	4–9	5–0	5–3	5–6	5–9	6–0	6–3
25	4–11⅜	5–2½	5–5⅝	5–8¾	5–11⅞	6–3	6–6⅛
26	5–1¾	5–5	5–8¼	5–11½	6–2¾	6–6	6–9¼
27	5–4⅛	5–7½	5–10⅞	6–2¼	6–5⅝	6–9	7–⅜
28	5–6½	5–10	6–1½	6–5	6–8½	7–0	7–3½
29	5–8⅞	6–½	6–4⅛	6–7¾	6–11⅜	7–3	7–6⅝
30	5–11¼	6–3	6–6¾	6–10½	7–2¼	7–6	7–9¾
31	6–1⅝	6–5½	6–9⅜	7–1¼	7–5⅛	7–9	8–⅞
32	6–4	6–8	7–0	7–4	7–8	8–0	8–4

TABLE XXII—(*Continued*)

MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	3¼	3⅜	3½	3⅝	3¾	4	4¼
	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″
2	0– 6½	0– 6¾	0– 7	0– 7¼	0– 7½	0– 8	0– 8½
3	0– 9¾	0–10⅛	0–10½	0–10⅞	0–11¼	1– 0	1– ¾
4	1– 1	1– 1½	1– 2	1– 2½	1– 3	1– 4	1– 5
5	1– 4¼	1– 4⅞	1– 5½	1– 6⅛	1– 6¾	1– 8	1– 9¼
6	1– 7½	1– 8¼	1– 9	1– 9¾	1–10½	2– 0	2– 1½
7	1–10¾	1–11⅝	2– ½	2– 1⅜	2– 2¼	2– 4	2– 5¾
8	2– 2	2– 3	2– 4	2– 5	2– 6	2– 8	2–10
9	2– 5¼	2– 6⅜	2– 7½	2– 8⅝	2– 9¾	3– 0	3– 2¼
10	2– 8½	2– 9¾	2–11	3– ¼	3– 1½	3– 4	3– 6½
11	2–11¾	3– 1⅛	3– 2½	3– 3⅞	3– 5¼	3– 8	3–10¾
12	3– 3	3– 4½	3– 6	3– 7½	3– 9	4– 0	4– 3
13	3– 6¼	3– 7⅞	3– 9½	3–11⅛	4– ¾	4– 4	4– 7¼
14	3– 9½	3–11¼	4– 1	4– 2¾	4– 4½	4– 8	4–11½
15	4– ¾	4– 2⅝	4– 4½	4– 6⅜	4– 8¼	5– 0	5– 3¾
16	4– 4	4– 6	4– 8	4–10	5– 0	5– 4	5– 8
17	4– 7¼	4– 9⅜	4–11½	5– 1⅝	5– 3¾	5– 8	6– ¼
18	4–10½	5– ¾	5– 3	5– 5¼	5– 7½	6– 0	6– 4½
19	5– 1¾	5– 4⅛	5– 6½	5– 8⅞	5–11¼	6– 4	6– 8¾
20	5– 5	5– 7½	5–10	6– ½	6– 3	6– 8	7– 1
21	5– 8¼	5–10⅞	6– 1½	6– 4⅛	6– 6¾	7– 0	7– 5¼
22	5–11½	6– 2¼	6– 5	6– 7¾	6–10½	7– 4	7– 9½
23	6– 2¾	6– 5⅝	6– 8½	6–11⅜	7– 2¼	7– 8	8– 1¾
24	6– 6	6– 9	7– 0	7– 3	7– 6	8– 0	8– 6
25	6– 9¼	7– ⅜	7– 3½	7– 6⅝	7– 9¾	8– 4	8–10¼
26	7– ½	7– 3¾	7– 7	7–10¼	8– 1½	8– 8	9– 2½
27	7– 3¾	7– 7⅛	7–10½	8– 1⅞	8– 5¼	9– 0	9– 6¾
28	7– 7	7–10½	8– 2	8– 5½	8– 9	9– 4	9–11
29	7–10¼	8– 1⅞	8– 5½	8– 9⅛	9– ¾	9– 8	10– 3¼
30	8– 1½	8– 5¼	8– 9	9– ¾	9– 4½	10– 0	10– 7½
31	8– 4¾	8– 8⅝	9– ½	9– 4⅜	9– 8¼	10– 4	10–11¾
32	8– 8	9– 0	9– 4	9– 8	10– 0	10– 8	11– 4

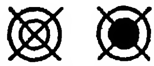
TABLE XXII—(*Continued*)

MULTIPLICATION TABLE FOR RIVET SPACING

Number of Spaces	Pitch of Rivets, in Inches						
	4½	4¾	5	5¼	5½	5¾	6
	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″	′ ″
2	0– 9	0– 9½	0–10	0–10½	0–11	0–11½	1– 0
3	1– 1½	1– 2¼	1– 3	1– 3¾	1– 4½	1– 5¼	1– 6
4	1– 6	1– 7	1– 8	1– 9	1–10	1–11	2– 0
5	1–10½	1–11¾	2– 1	2– 2¼	2– 3½	2– 4¾	2– 6
6	2– 3	2– 4½	2– 6	2– 7½	2– 9	2–10½	3– 0
7	2– 7½	2– 9¼	2–11	3– ¾	3– 2½	3– 4¼	3– 6
8	3– 0	3– 2	3– 4	3– 6	3– 8	3–10	4– 0
9	3– 4½	3– 6¾	3– 9	3–11¼	4– 1½	4– 3¾	4– 6
10	3– 9	3–11½	4– 2	4– 4½	4– 7	4– 9½	5– 0
11	4– 1½	4– 4¼	4– 7	4– 9¾	5– ½	5– 3¼	5– 6
12	4– 6	4– 9	5– 0	5– 3	5– 6	5– 9	6– 0
13	4–10½	5– 1¾	5– 5	5– 8¼	5–11½	6– 2¾	6– 6
14	5– 3	5– 6½	5–10	6– 1½	6– 5	6– 8½	7– 0
15	5– 7½	5–11¼	6– 3	6– 6¾	6–10½	7– 2¼	7– 6
16	6– 0	6– 4	6– 8	7– 0	7– 4	7– 8	8– 0
17	6– 4½	6– 8¾	7– 1	7– 5¼	7– 9½	8– 1¾	8– 6
18	6– 9	7– 1½	7– 6	7–10½	8– 3	8– 7½	9– 0
19	7– 1½	7– 6¼	7–11	8– 3¾	8– 8½	9– 1¼	9– 6
20	7– 6	7–11	8– 4	8– 9	9– 2	9– 7	10– 0
21	7–10½	8– 3¾	8– 9	9– 2¼	9– 7½	10– ¾	10– 6
22	8– 3	8– 8½	9– 2	9– 7½	10– 1	10– 6½	11– 0
23	8– 7½	9– 1¼	9– 7	10– ¾	10– 6½	11– ¼	11– 6
24	9– 0	9– 6	10– 0	10– 6	11– 0	11– 6	12– 0
25	9– 4½	9–10¾	10– 5	10–11¼	11– 5½	11–11¾	12– 6
26	9– 9	10– 3½	10–10	11– 4½	11–11	12– 5½	13– 0
27	10– 1½	10– 8¼	11– 3	11– 9¾	12– 4½	12–11¼	13– 6
28	10– 6	11– 1	11– 8	12– 3	12–10	13– 5	14– 0
29	10–10½	11– 5¾	12– 1	12– 8¼	13– 3½	13–10¾	14– 6
30	11– 3	11–10½	12– 6	13– 1½	13– 9	14– 4½	15– 0
31	11– 7½	12– 3¼	12–11	13– 6¾	14– 2½	14–10¼	15– 6
32	12– 0	12– 8	13– 4	14– 0	14– 8	15– 4	16– 0

TABLE XXIII

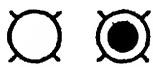
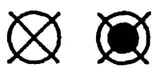
CONVENTIONAL SIGNS FOR STRUCTURAL RIVETS

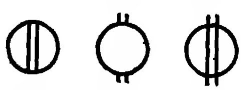
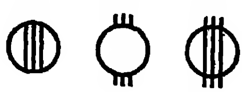
	Shop / Field
Two full heads	
Countersunk inside and not chipped	
Countersunk outside and not chipped	
Countersunk both sides and not chipped	

Where rivets must be chipped, the drawings should state the fact.

	Inside / Outside / Both Sides
Flatten to ⅜ inch high	

(a)

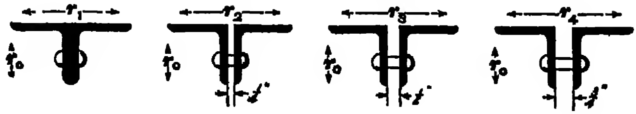
	Shop / Field
Two full heads	
Countersunk inside and chipped	
Countersunk outside and chipped	
Countersunk both sides and chipped	

	Inside / Outside / Both Sides
Flatten to ⅛ inch high or countersunk and not chipped	
Flatten to ¼ inch high	
Flatten to ⅜ inch high	

(b)

TABLE XXIV

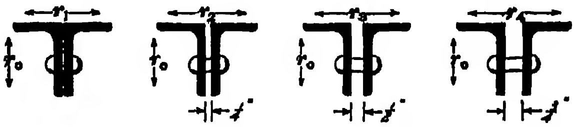
RADII OF GYRATION FOR TWO ANGLES HAVING EQUAL LEGS AND PLACED BACK TO BACK



Dimensions Inches	Thickness Inches	Area of Two Angles Square Inches	Radii of Gyration				
			r_0	r_1	r_2	r_3	r_4
2×2	$\frac{3}{16}$	1.42	.62	.84	.93	1.03	1.13
2×2	$\frac{7}{16}$	3.12	.59	.88	.98	1.08	1.19
2½×2½	$\frac{3}{16}$	1.80	.78	1.04	1.13	1.22	1.32
2½×2½	$\frac{1}{2}$	4.50	.74	1.10	1.19	1.29	1.40
3×3	$\frac{1}{4}$	2.88	.93	1.25	1.34	1.43	1.53
3×3	$\frac{5}{8}$	6.72	.88	1.32	1.41	1.51	1.62
3½×3½	$\frac{5}{16}$	4.18	1.08	1.47	1.56	1.65	1.74
3½×3½	$\frac{13}{16}$	10.06	1.02	1.55	1.64	1.74	1.85
4×4	$\frac{5}{16}$	4.80	1.24	1.67	1.76	1.85	1.94
4×4	$\frac{13}{16}$	11.68	1.18	1.75	1.84	1.94	2.04
6×6	$\frac{3}{8}$	8.72	1.88	2.49	2.58	2.67	2.76
6×6	1	22.00	1.80	2.59	2.68	2.77	2.87
8×8	$\frac{1}{2}$	15.50	2.50	3.32	3.40	3.49	3.58
8×8	$1\frac{1}{8}$	33.46	2.42	3.42	3.51	3.60	3.69

TABLE XXV

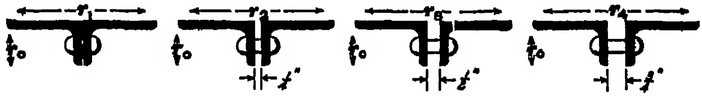
RADII OF GYRATION FOR TWO ANGLES HAVING UNEQUAL LEGS AND PLACED BACK TO BACK



Dimensions Inches	Thickness Inch	Area of Two Angles Square Inches	Radii of Gyration				
			r_0	r_1	r_2	r_3	r_4
2½×2	3/16	1 62	.79	.79	88	97	1 07
2½×2	½	4 00	75	84	94	1 04	1 15
3×2½	¼	2 62	.95	1 00	1 09	1 18	1 28
3×2½	9/16	5 56	91	1 05	1.15	1 25	1 35
3½×2½	¼	2 88	1 12	96	1 04	1 13	1 23
3½×2½	11/16	7 10	1 06	1 03	1.13	1 23	1.33
3½×3	5/16	3 86	1 10	1 22	1 31	1 40	1 49
3½×3	13/16	9 24	1 04	1 30	1 40	1 50	1 60
4×3	5/16	4 18	1 27	1 17	1.26	1 35	1 44
4×3	13/16	10 06	1 21	1 25	1 35	1 45	1 55
5×3	5/16	4 80	1 61	1 09	1 17	1.26	1.35
5×3	13/16	11 68	1 55	1 18	1 27	1 37	1.47
5×3½	5/16	5 12	1 61	1 33	1 41	1 50	1 59
5×3½	7/8	13 34	1 53	1 42	1 51	1 61	1 71
6×3½	3/8	6 84	1 94	1 26	1.34	1 43	1.53
6×3½	1	17 00	1 85	1 37	1 46	1.56	1 67
6×4	3/8	7 22	1 93	1 50	1 58	1 67	1 76
6×4	1	18 00	1 85	1 60	1 69	1 79	1.89
7×3½	7/16	8 80	2.26	1 21	1 29	1 39	1 47
7×3½	1	19 00	2 19	1 31	1.40	1 50	1.60

TABLE XXVI

RADII OF GYRATION FOR TWO ANGLES HAVING UNEQUAL LEGS AND PLACED BACK TO BACK



Dimensions Inches	Thickness Inch	Area of Two Angles Square Inches	Radii of Gyration				
			r_0	r_1	r_2	r_3	r_4
2½×2	3/16	1 62	60	1 10	1 19	1 28	1 39
2½×2	½	4 00	56	1 16	1 25	1.35	1 46
3×2½	¼	2 62	75	1 31	1 40	1 50	1 59
3×2½	9/16	5 56	.72	1 37	1 46	1 56	1 66
3½×2½	¼	2 88	74	1 58	1 67	1 76	1 86
3½×2½	11/16	7 10	67	1 66	1 76	1 86	1.96
3½×3	5/16	3 86	.90	1 52	1 61	1 71	1 80
3½×3	13/16	9.24	.85	1 61	1 71	1 81	1 91
4×3	5/16	4.18	89	1 79	1 88	1.97	2 07
4×3	13/16	10.06	83	1 88	1.98	2.08	2 18
5×3	5/16	4 80	85	2 33	2 42	2.51	2 61
5×3	13/16	11 68	.80	2 42	2 52	2 62	2 72
5×3½	5/16	5 12	1 03	2 26	2 35	2 44	2 54
5×3½	⅞	13 34	.96	2 36	2 45	2 55	2 65
6×3½	⅜	6.84	.99	2 81	2 90	3 00	3 10
6×3½	1	17.00	.92	2 93	3 03	3.13	3 23
6×4	⅜	7 22	1 17	2.74	2.83	2.92	3.01
6×4	1	18 00	1 09	2.85	2.94	3.04	3 14
7×3½	7/16	8 80	.95	3.37	3 46	3 56	3 66
7×3½	1	19.00	.89	3 48	3 58	3 68	3 78

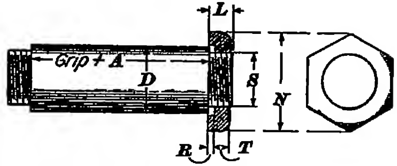
TABLE XXVII

AREAS TO BE DEDUCTED FOR RIVET HOLES

Thickness of Plate Inches	Diameter of Rivets, in Inches				
	1/2	5/8	3/4	7/8	1
1/4	156	188	219	.250	.281
5/16	195	.234	.273	.313	.352
3/8	.234	281	328	.375	.422
7/16	.273	.328	.383	.438	492
1/2	313	375	438	500	563
9/16	.352	.422	.492	563	.633
5/8	.391	469	547	.625	.703
11/16	430	516	602	688	.773
3/4	469	563	656	.750	844
13/16	508	.609	.711	.813	.914
7/8	547	.656	766	.875	.984
15/16	.586	.703	.820	.938	1.055
1	625	.750	.875	1 000	1.125
1 1/8	703	844	.984	1 125	1 266
1 1/4	.781	.938	1.094	1.250	1.406
1 3/8	859	1 031	1 203	1 375	1 547
1 1/2	.937	1.125	1.313	1 500	1 688
Counter-sunk rivets	031	.059	.082	.109	.141

TABLE XXVIII

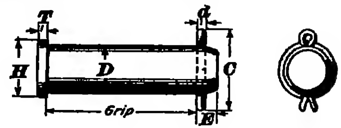
DIMENSIONS OF SCREW ENDS AND NUTS FOR PINS, IN INCHES



Diameter of Pin D	Diameter of Screw S	Length of Screw L	Thickness of Nut T	Recess in Nut R	Long Diameter of Nut N	Weight of Nut Pounds	Add to Grip A
2 to $2\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{1}{2}$	1	$\frac{3}{8}$	$3\frac{3}{4}$	2 5	$\frac{1}{4}$
$2\frac{3}{8}$ to $2\frac{3}{4}$	2	$1\frac{1}{2}$	1	$\frac{3}{8}$	$4\frac{5}{16}$	2 5	$\frac{1}{4}$
$2\frac{7}{8}$ to $3\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{2}$	1	$\frac{3}{8}$	$5\frac{3}{16}$	3 0	$\frac{1}{4}$
$3\frac{5}{8}$ to 4	3	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$5\frac{3}{4}$	5 5	$\frac{1}{2}$
$4\frac{1}{8}$ to $4\frac{3}{4}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$6\frac{5}{8}$	7 0	$\frac{1}{2}$
$4\frac{7}{8}$ to $5\frac{1}{4}$	4	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$7\frac{1}{2}$	8 5	$\frac{1}{2}$
$5\frac{3}{8}$ to 6	$4\frac{1}{2}$	$1\frac{7}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$	$8\frac{1}{8}$	11 0	$\frac{1}{2}$
$6\frac{1}{8}$ to $6\frac{1}{2}$	5	$2\frac{3}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	9	12 0	$\frac{3}{4}$
$6\frac{5}{8}$ to $7\frac{1}{2}$	$5\frac{1}{2}$	$2\frac{3}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	$9\frac{1}{2}$	13 5	$\frac{3}{4}$
$7\frac{5}{8}$ to 8	6	$2\frac{3}{8}$	$1\frac{1}{2}$	$\frac{3}{4}$	$10\frac{3}{8}$	17 0	$\frac{3}{4}$

TABLE XXIX

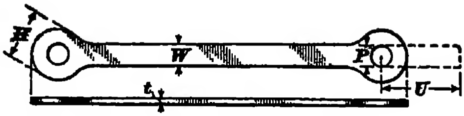
DIMENSIONS OF LATERAL PINS, IN INCHES



Diameter of Pin D	Diameter of Head H	Thickness of Head T	Diameter of Cotter Pin d	Length of Cotter Pin C	Length of End E
1	$1\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$1\frac{3}{4}$	$\frac{5}{8}$
$1\frac{1}{4}$	$1\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	2	$\frac{5}{8}$
$1\frac{1}{2}$	$1\frac{3}{4}$	$\frac{1}{4}$	$\frac{5}{16}$	$2\frac{1}{2}$	$\frac{7}{8}$
$1\frac{3}{4}$	2	$\frac{1}{4}$	$\frac{5}{16}$	$2\frac{3}{4}$	$\frac{7}{8}$
2	$2\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	3	1
$2\frac{1}{4}$	$2\frac{5}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$3\frac{1}{4}$	1
$2\frac{1}{2}$	$2\frac{7}{8}$	$\frac{3}{8}$	$\frac{7}{16}$	$3\frac{3}{4}$	$1\frac{1}{8}$
$2\frac{3}{4}$	$3\frac{1}{8}$	$\frac{3}{8}$	$\frac{7}{16}$	4	$1\frac{1}{8}$
3	$3\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	5	$1\frac{3}{8}$

TABLE XXX

STEEL EYEBARS

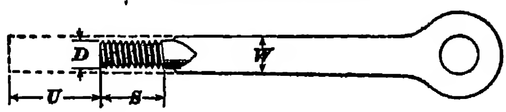


Width of Body of Bar W Inches	Minimum Thickness of Bar t Inch	Diameter of Head of Bar H Inches	Diameter of Largest Pinhole P Inches	Additional Length to Form Head U
				′ ″
2	½	4½	1⅞	0– 7½
2	½	5½	2⅞	1– ½
2½	½	5½	2⅛	0– 9½
2½	½	6½	3⅛	1– 1½
3	¾	6½	2½	0–10½
3	¾	8	4	1– 5½
3	¾	9	5	1–10½
4	¾	9½	4⅛	1– 5½
4	¾	10½	5⅛	1– 9
4	¾	11½	6⅛	2– 3½
5	¾	11½	4⅝	1– 8
5	¾	12½	5⅝	2– 0
5	1	13	6⅛	2– 3½
5	1	14	7⅛	2– 8
6	⅞	13½	5¼	1– 9½
6	⅞	14½	6¼	2– 3
6	1	15½	7¼	2– 7½
7	15/16	15½	5⅝	2– 2
7	15/16	17	7⅛	2– 8
8	1	17	5¾	2– 1½
8	1	18	6¾	2– 6½
8	1	19	8	2–11

TABLE XXXI

ADJUSTABLE EYEBARS

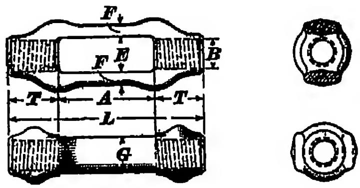
(*All dimensions in inches*)



Width of Bar W	Thickness of Bar t	Diameter of Upset End D	Length of Screw S	Additional Length Required to Form Screw End U
2	1	2	5½	6
3	⅞	2¼	6½	11½
3	1	2½	6½	11¼
3	1¼	2¾	7	11
3	1½	3	7	10
4	¾	2½	6½	12¾
4	1	2¾	7	11
4	1¼	3¼	7¼	11
4	1½	3½	7½	10
4	1¾	3¾	7½	9½
5	¾	2¾	7	11
5	1	3¼	7¼	10½
5	1¼	3½	7½	9½
5	1½	3¾	7½	9¾
5	1¾	4	7½	10
6	1⅛	3¾	7½	10
6	1¼	3¾	7½	9
6	1½	4	7½	9

TABLE XXXII

PRESSED WROUGHT-IRON TURNBUCKLES



Diameter of Screw Inches	Dimensions, in Inches					Weight of One Turnbuckle Pounds
B	*L*	*T*	*E*	*F*	*G*	
7/8	8 5/8	1 5/16	1 1/4	3/8	1 1/4	2 1/2
1	9	1 1/2	1 5/16	7/16	1 1/4	3 1/2
1 1/8	9 3/8	1 11/16	1 7/16	1/2	1 1/4	4
1 1/4	9 3/4	1 7/8	1 9/16	1/2	1 1/2	5 1/4
1 3/8	10 1/8	2 1/16	1 11/16	1/2	1 5/8	6
1 1/2	10 1/2	2 1/4	1 3/4	5/8	1 3/4	7
1 5/8	10 7/8	2 7/16	2	5/8	1 3/4	8 1/2
1 3/4	11 1/4	2 5/8	2 1/8	5/8	2	10
1 7/8	11 5/8	2 13/16	2 3/16	11/16	2	11 1/2
2	12	3	2 3/8	11/16	2 1/4	13
2 1/8	12 3/8	3 3/16	2 1/2	23/32	2 1/2	15
2 1/4	12 3/4	3 3/8	2 11/16	13/16	2 1/2	18
2 3/8	13 1/8	3 9/16	2 3/4	13/16	2 3/4	20
2 1/2	13 1/2	3 3/4	3 1/16	27/32	3	24
2 5/8	13 7/8	3 15/16	3 1/8	15/16	3	28
2 3/4	14 1/4	4 1/8	3 1/4	15/16	3 1/4	30
2 7/8	14 5/8	4 5/16	3 7/16	1 1/32	3 1/4	34
3	15	4 1/2	3 5/8	1 1/32	3 1/2	38
3 1/4	15 3/4	4 7/8	3 7/8	1 1/16	4	50
3 1/2	16 1/2	5 1/4	4 1/4	1 7/32	4	65
3 3/4	18	6	4 7/16	1 5/16	5	82
4	18	6	4 5/8	1 7/16	5	100

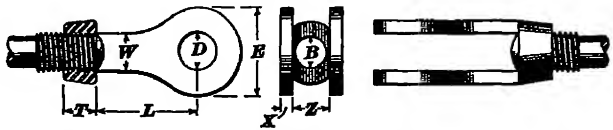
TABLE XXXIII

HEADS FOR LATERAL RODS

Diameter of Round or Side of Square Bar Inches *A*	Diameter of Head Inches *E*	Diameter of Largest Pinhole Inches *D*	Additional Length for One Head Round Bar Inches *U*	Additional Length for One Head Square Bar Inches *U*
1	4¼	2½	18	16
1⅛	4¼	2½	16	14
1¼	5	2¾	20½	18½
1⅜	5	2¾	18½	16½
1½	5½	3	20	18
1⅝	5½	3	18½	16½
1¾	6	3¼	21	18
1⅞	6	3¼	19½	16½
2	6½	3½	21½	18½
2⅛	6½	3½	20	17
2¼	7½	4	24½	21½
2⅜	7½	4	22¾	19¾
2½	8	4	25½	22½
2⅝	8	4	24	21
2¾	8	4	22½	19½

TABLE XXXIV

CLEVISES



Diameter of Screw Inches B	Length of Thread Inches T	Diameter of Head Inches E	Diameter of Largest Pin Inches D	Length of Fork Inches L	Width of Fork Inches Z	Width of Side Inches W	Thickness of Side Inch X	Weight of One Clevis Pounds
$\frac{7}{8}$ and 1	$1\frac{1}{2}$	4	$2\frac{1}{2}$	$5\frac{1}{4}$	$1\frac{1}{8}$	2	$\frac{1}{2}$	$5\frac{1}{2}$
$1\frac{1}{8}$ and $1\frac{1}{4}$	$1\frac{1}{2}$	4	$2\frac{1}{2}$	$5\frac{1}{4}$	$1\frac{3}{8}$	2	$\frac{1}{2}$	$6\frac{1}{2}$
$1\frac{3}{8}$ and $1\frac{1}{2}$	$1\frac{1}{2}$	4	2	$5\frac{1}{4}$	$1\frac{5}{8}$	2	$\frac{1}{2}$	$7\frac{1}{4}$
$1\frac{5}{8}$ and $1\frac{3}{4}$	$1\frac{3}{4}$	4	2	$5\frac{1}{4}$	$1\frac{7}{8}$	2	$\frac{5}{8}$ and $\frac{3}{4}$	9
$1\frac{7}{8}$ and 2	2	$4\frac{7}{8}$	$2\frac{1}{4}$	7	$2\frac{1}{8}$	2	$\frac{1}{4}$	$13\frac{3}{4}$
$2\frac{1}{8}$ and $2\frac{1}{4}$	$2\frac{1}{8}$	$5\frac{7}{8}$	$2\frac{1}{2}$	7	$2\frac{3}{8}$	$2\frac{1}{2}$	$\frac{3}{4}$	$20\frac{1}{4}$
$2\frac{3}{8}$ and $2\frac{1}{2}$	$2\frac{1}{4}$	$6\frac{3}{8}$	$2\frac{3}{4}$	7	$2\frac{5}{8}$	$2\frac{1}{2}$	$\frac{7}{8}$	$25\frac{1}{4}$
$2\frac{5}{8}$ and $2\frac{3}{4}$	$2\frac{3}{4}$	$6\frac{7}{8}$	3	$8\frac{3}{4}$	$2\frac{7}{8}$	3	1	30

TABLE XXXV

WORKING STRESSES FOR COMPRESSION

$\frac{l}{r}$	$\frac{16,000}{1+\frac{l^2}{18,000\,r^2}}$	$\frac{l}{r}$	$\frac{16,000}{1+\frac{l^2}{18,000\,r^2}}$	$\frac{l}{r}$	$\frac{16,000}{1+\frac{l^2}{18,000\,r^2}}$
16	15,780	51	13,980	86	11,340
17	15,750	52	13,910	87	11,260
18	15,720	53	13,840	88	11,190
19	15,690	54	13,770	89	11,110
20	15,650	55	13,700	90	11,030
21	15,620	56	13,630	91	10,960
22	15,580	57	13,550	92	10,880
23	15,530	58	13,480	93	10,810
24	15,500	59	13,410	94	10,730
25	15,460	60	13,330	95	10,660
26	15,420	61	13,260	96	10,580
27	15,380	62	13,180	97	10,510
28	15,330	63	13,110	98	10,430
29	15,290	64	13,030	99	10,360
30	15,240	65	12,960	100	10,290
31	15,190	66	12,880	101	10,210
32	15,140	67	12,810	102	10,140
33	15,090	68	12,730	103	10,070
34	15,030	69	12,650	104	9,990
35	14,980	70	12,580	105	9,920
36	14,930	71	12,500	106	9,850
37	14,870	72	12,420	107	9,780
38	14,810	73	12,340	108	9,710
39	14,750	74	12,270	109	9,640
40	14,690	75	12,190	110	9,570
41	14,630	76	12,110	111	9,500
42	14,570	77	12,040	112	9,430
43	14,510	78	11,960	113	9,360
44	14,450	79	11,880	114	9,290
45	14,380	80	11,800	115	9,220
46	14,320	81	11,730	116	9,160
47	14,250	82	11,650	117	9,090
48	14,180	83	11,570	118	9,020
49	14,120	84	11,490	119	8,950
50	14,050	85	11,420	120	8,890

TABLE XXXVI

WORKING STRESSES FOR SHEAR

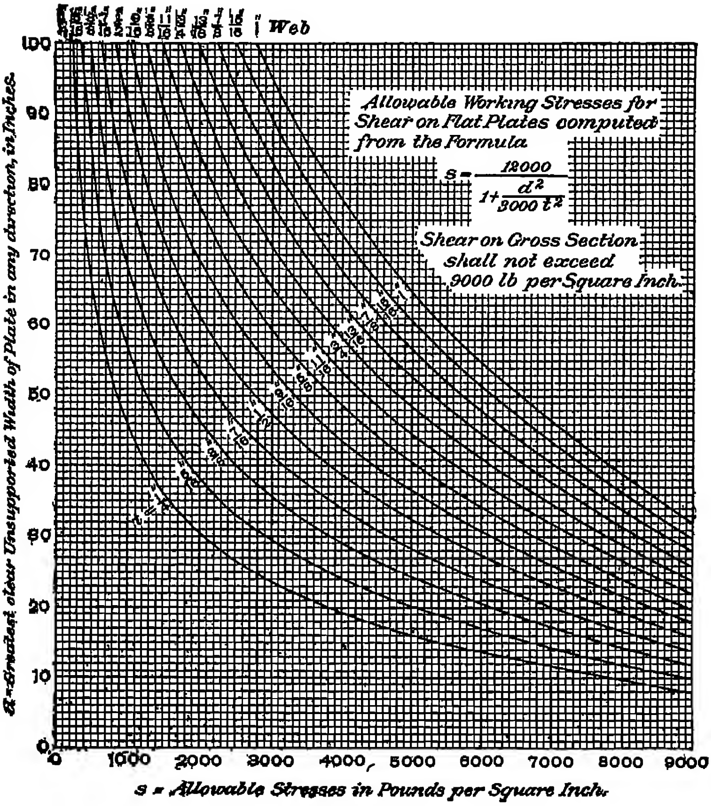


TABLE XXXVII

SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diameter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 6,000 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 12,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$
$\frac{1}{2}$	1963	1,180	2,360	1,500	1,880	2,250	2,630	3,000							
$\frac{5}{8}$	.3068	1,840	3,680	1,880	2,340	2,810	3,280	3,750	4,220	4,690					
$\frac{3}{4}$	.4418	2,650	5,300	2,250	2,810	3,380	3,940	4,500	5,060	5,630	6,190	6,750			
$\frac{7}{8}$	6013	3,610	7,220	2,630	3,280	3,940	4,590	5,250	5,910	6,560	7,220	7,880	8,530	9,190	
1	7854	4,710	9,420	3,000	3,750	4,500	5,250	6,000	6,750	7,500	8,250	9,000	9,750	10,500	11,250

TABLE XXXVIII

SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diameter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 7,500 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 15,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$
$\frac{1}{2}$	1963	1,470	2,940	1,880	2,340	2,810	3,280	3,750							
$\frac{5}{8}$	.3068	2,300	4,600	2,340	2,930	3,520	4,100	4,690	5,280	5,860					
$\frac{3}{4}$	.4418	3,310	6,630	2,810	3,520	4,220	4,920	5,630	6,330	7,030	7,720	8,440			
$\frac{7}{8}$	6013	4,510	9,020	3,280	4,100	4,920	5,740	6,560	7,380	8,200	9,030	9,850	10,670	11,480	
1	7854	5,890	11,780	3,750	4,690	5,630	6,560	7,500	8,440	9,380	10,310	11,250	12,190	13,130	14,060

TABLE XXXIX

SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diameter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 9,000 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 18,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$
$\frac{1}{2}$	1963	1,770	3,530	2,250	2,810	3,380	3,940	4,500							
$\frac{5}{8}$	3068	2,760	5,520	2,810	3,520	4,220	4,920	5,630	6,330	7,030					
$\frac{3}{4}$	4418	3,980	7,950	3,380	4,220	5,060	5,910	6,750	7,590	8,440	9,280	10,130			
$\frac{7}{8}$	6013	5,410	10,820	3,940	4,920	5,910	6,890	7,880	8,860	9,840	10,830	11,810	12,800	13,780	
1	.7854	7,070	14,140	4,500	5,630	6,750	7,880	9,000	10,130	11,250	12,380	13,500	14,630	15,750	16,880

TABLE XL

SHEARING AND BEARING VALUES OF RIVETS, IN POUNDS

Diameter of Rivet Inch	Area of Rivet Square Inch	Shear Values at 11,000 Pounds per Square Inch		Bearing Values for Different Thicknesses of Plate, in Inches, at 22,000 Pounds per Square Inch											
		Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$
$\frac{1}{2}$	1963	2,160	4,320	2,750	3,440	4,130	4,820	5,500							
$\frac{5}{8}$	3068	3,370	6,750	3,440	4,300	5,160	6,020	6,880	7,740	8,600					
$\frac{3}{4}$	4418	4,860	9,720	4,130	5,160	6,190	7,220	8,250	9,280	10,320	11,340	12,380			
$\frac{7}{8}$	6013	6,610	13,230	4,810	6,020	7,220	8,430	9,630	10,840	12,040	13,240	14,440	15,640	16,840	
1	7854	8,640	17,280	5,500	6,880	8,250	9,630	11,000	12,380	13,750	15,130	16,500	17,880	19,250	20,630

TABLE XLI

BEARING VALUES AND RESISTING MOMENTS OF PINS

Diameter of Pin Inches	Area of Pin Square Inches	Bearing Values, in Pounds, for 1 Inch Thickness of Plate			Moments, in Inch-Pounds, for Extreme Fiber Stresses of				
		15,000 Pounds per Square Inch	18,000 Pounds per Square Inch	22,000 Pounds per Square Inch	15,000 Pounds per Square Inch	18,000 Pounds per Square Inch	20,000 Pounds per Square Inch	22,000 Pounds per Square Inch	25,000 Pounds per Square Inch
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*
1	.785	15,000	18,000	22,000	1,470	1,770	1,960	2,160	2,450
$1\frac{1}{4}$	1 227	18,750	22,500	27,500	2,880	3,450	3,830	4,220	4,790
$1\frac{1}{2}$	1 767	22,500	27,000	33,000	4,970	5,960	6,630	7,290	8,280
$1\frac{3}{4}$	2 405	26,250	31,500	38,500	7,890	9,470	10,500	11,570	13,200
2	3 142	30,000	36,000	44,000	11,800	14,100	15,700	17,280	19,600
$2\frac{1}{8}$	3 547	31,880	38,250	46,750	14,100	17,000	18,800	20,730	23,600
$2\frac{1}{4}$	3 976	33,750	40,500	49,500	16,800	20,100	22,400	24,600	28,000
$2\frac{3}{8}$	4.430	35,630	42,750	52,250	19,700	23,700	26,300	28,900	32,900
$2\frac{1}{2}$	4 909	37,500	45,000	55,000	23,000	27,600	30,700	33 700	38,400
$2\frac{5}{8}$	5 412	39,380	47,250	57,750	26,600	32,000	35,500	39,000	44,400
$2\frac{3}{4}$	5 940	41,250	49,500	60,500	30,600	36,800	40,800	44,900	51,000
$2\frac{7}{8}$	6 492	43,130	51,750	63,250	35,000	42,000	46,700	51,300	58,300
3	7 069	45,000	54,000	66,000	39,800	47,700	53,000	58,300	66,300
$3\frac{1}{8}$	7 670	46,880	56,250	68,750	44,900	53,900	59,900	65,900	74,900
$3\frac{1}{4}$	8 296	48,750	58,500	71,500	50,600	60,700	67,400	74,100	84,300
$3\frac{3}{8}$	8 946	50,630	60,750	74,250	56,600	67,900	75,500	83,000	94,400

3⅛	9 621	52,500	63,000	77,000	63,100	75,800	84,200	92,600	105,200
3¼	10 32	54,380	65,250	79,750	70,100	84,200	93,500	102,900	116,900
3⅜	11 05	56,250	67,500	82,500	77,700	93,200	103,500	113,900	129,400
3⅞	11 79	58,130	69,750	85,250	85,700	102,800	114,200	125,600	142,800
4	12 57	60,000	72,000	88,000	94,200	113,100	125,700	138,200	157,100
4⅛	13 36	61,880	74,250	90,750	103,400	124,000	137,800	151,600	172,300
4¼	14.19	63,750	76,500	93,500	113,000	135,700	150,700	165,800	188,400
4⅜	15 03	65,630	78,750	96,250	123,300	148,000	164,400	180,800	205,500
4½	15 90	67,500	81,000	99,000	134,200	161,000	178,900	196,800	223,700
4⅝	16 80	69,380	83,250	101,750	145,700	174,800	194,300	213,700	242,800
4¾	17 72	71,250	85,500	104,500	157,800	189,400	210,400	231,500	263,000
4⅞	18 67	73,130	87,750	107,250	170,600	204,700	227,500	250,200	284,400
5	19 64	75,000	90,000	110,000	184,100	220,900	245,400	270,000	306,800
5¼	21 65	78,750	94,500	115,500	213,100	255,700	284,100	312,500	355,200
5½	23 76	82,500	99,000	121,000	245,000	294,000	326,700	359,300	408,300
5¾	25 97	86,250	103,500	126,500	280,000	335,900	373,300	410,600	466,600
6	28 27	90,000	108,000	132,000	318,100	381,700	424,100	466,500	530,200
6¼	30 68	93,750	112,500	137,500	359,500	431,400	479,400	527,300	599,200
6½	33 18	97,500	117,000	143,000	404,400	485,400	539,200	593,100	674,000
6¾	35 79	101,250	121,500	148,500	452,900	543,500	603,900	664,200	754,800
7	38 48	105,000	126,000	154,000	505,100	606,100	673,500	740,800	841,900
7¼	41 28	108,750	130,500	159,500	561,200	673,400	748,200	823,000	935,300
7½	44.18	112,500	135,000	165,000	621,300	745,500	828,400	911,200	1,035,400
7¾	47 17	116,250	139,500	170,500	685,500	822,600	914,000	1,005,300	1,142,500
8	50 27	120,000	144,000	176,000	754,000	904,800	1,005,300	1,105,800	1,256,600

BRIDGE SPECIFICATIONS

INTRODUCTION

1. In the early days of bridge trusses, when wood, being plentiful and cheap, was used to a great extent, bridges were invariably built without plans, and in a great many cases without previously calculating any stresses or designing any members. They were built by men called *bridge carpenters*, and the experience of the foreman or superintendent of the gang usually enabled him to decide on the sizes of members to be used As the loads were then light and timber was much more plentiful than it is now, the errors were generally on the side of safety; that is, the bridges were made more than strong enough for the loads.

2. When wood was replaced by wrought iron, it became necessary to manufacture in the shops most of the members, and designs and plans were made. There were no systematic scientific methods of design, however; the details, instead of being proportioned according to the forces they were to resist, were designed and arranged as seemed most convenient. Many bridges at that time were designed by shop foremen that had no knowledge of stresses; the members were laid out full size on the shop floors, and made so as to utilize the material on hand in the stock yard. Very few, if any, bridges were designed to allow an increase in the loads they were to carry in the future.

3. Later, engineers began to realize that both safety and economy required that bridges should be designed according to scientific principles and constructed in conformity with

fixed rules derived from both experience and theoretical investigations. Such rules, when assembled together for the guidance of the designer, builder, or contractor, are called **specifications.** Specifications differed greatly at first, but after a short time they began to approach each other, and today the points in which the various specifications differ from one another are comparatively few. It is not unlikely that standard specifications for the construction of all bridges of the same type will be adopted in the near future. The introduction of such uniform system will greatly facilitate bridge design and construction.

4. When the earlier bridges were finished, the plans, if any, that had been used in the design and construction were either destroyed or lost, as the importance of saving them for future reference was apparently not fully realized. As a result, there are at the present time many bridges in use for which no plans can be found; when it is desired to know if they can support with safety heavier loads than they have been carrying, it is difficult and very expensive to calculate their strength, for it is first necessary to measure accurately the span, panel length, and depth of girders, and trusses, the cross-sections of stringers, floorbeams, girders, and truss members, and the details of all connections. For this reason, it has become the custom to keep on file detail plans of every new bridge; these plans show the location of every rivet and the size of every piece of metal in the structure, and are of great value for future reference.

5. In the following articles are given bridge specifications agreeing with the best practice in the United States at the present time. The clauses are those that actual practice has shown to be most suitably adapted to the purposes stated. Some of these specifications will be discussed at the end of this Section; others, such as those relating to plate girders, will be given in the Sections on design. It will be sufficient for the student to read these specifications very carefully, so as to get a good idea of their contents; it is not necessary to memorize them.

The bridges in the following Sections will be designed according to these specifications. In case it is necessary to design bridges according to other specifications, as is usually the case when a designer works for a bridge or railroad company, it will simply be necessary to read over the other specifications and design the work accordingly.

SPECIFICATIONS FOR THE DESIGN OF STEEL BRIDGES

PLANS AND PROPOSALS

6. **Engineer and Contractor.**—The term **Engineer,** where used in these specifications, refers to the Chief or Consulting Engineer in charge of the design and construction of the bridge, and to his duly authorized assistants or representatives. The term **Contractor** refers to the bidder to whom the contract for the work has been awarded, and to his duly authorized representatives. The decision of the Engineer shall be authoritative in all cases of uncertainty.

7. **Letter of Invitation.**—A copy of these specifications will be furnished each bidder; in addition, he will be given a **letter of invitation** to bid, stating the general description of the work for which bids are desired and any additional facts that may be necessary. In case the requirements given in the letter of invitation conflict in any way with those in the specifications, those in the letter of invitation will rule.

8. **Bids.**—Bids shall state the total sum for which the work, as described in the letter of invitation, will be done, the estimated weight, and price per pound, of each class of material, and the amount of time required to complete the work. They shall be made with the understanding that the Engineer reserves the right to make such changes in the plans, before the commencement of work, as may be considered advisable by him to render the bridge a satisfactory piece of work. The increase or decrease in price due

to such change shall be estimated from the pound price of the original bid, and shall be added to or deducted from the contract price.

9. **Extension of Time.**—The Contractor shall be responsible for damages on account of delay from any cause during the progress of the work. If any unforeseen delay shall arise, it will entitle him to an extension of time, to be granted in writing by the Engineer at the time of the delay.

10. **Patent Devices.**—The Contractor shall assume all responsibility for the use of patent devices in any part of the bridge, or in connection with the work of construction.

11. **Subcontractors.**—No part of the work shall be sublet, nor shall the contract for the whole or any part of the work be assigned by the Contractor, without the written consent of the Engineer. No part of the work shall be done in a shop not properly equipped with modern facilities. These specifications shall be binding on subcontractors in every respect.

12. **Plans and Stress Sheets.**—As a rule, the Engineer will provide each bidder with plans and stress sheets, showing the loads assumed, the resulting stresses, the proposed sizes and sectional areas of the members, and the style of the details and connections, as well as lengths, heights, and clearances. The bidder shall verify the plans before he submits his bid, and he alone shall be responsible for any errors, except as to general layout. He shall return the Engineer's plans if his bid is not accepted. If the Engineer does not furnish plans and stress sheets as described, the bidder shall furnish them with his bid, if requested to do so by the Engineer.

13. **Working Drawings.**—After a contract has been awarded and before any material is ordered or work commenced, the Contractor shall submit to the Engineer three complete sets of working drawings, including erection diagrams. When satisfactory, one set of such drawings and diagrams will be approved and returned to the Contractor, and all work shall be done in accordance with them. The

Contractor alone shall be responsible for the correctness of these drawings, even if they have been approved by the Engineer. No changes shall be made on the drawings after they have been approved, unless authorized in writing by the Engineer. On the completion of the work, the Contractor shall furnish the Engineer one complete set of tracings of the working drawings, which will be permanently filed in the office of the Engineer. The Contractor shall, when required, furnish also the necessary plans for designing the masonry.

Drawings shall preferably be not more than 24 in. $\times$ 36 in., with details drawn to a scale of $\frac{3}{4}$ or 1 inch to 1 foot.

DESIGN OF RAILROAD BRIDGES

GENERAL DIMENSIONS

14. Kinds of Bridges.—The following kinds of bridges shall preferably be used:

For spans less than 25 feet in length, rolled beams.

For spans from 25 to 100 feet in length, plate girders.

For spans from 100 to 150 feet in length, riveted trusses.

For spans over 150 feet in length, pin-connected trusses.

If, for any reason, it is desired to depart more than 10 feet from these limits, permission in writing must be obtained from the Engineer.

Deck bridges will have the preference wherever the conditions permit their use.

15. Panel Lengths and Depths.—The depth of plate girders shall preferably be one-eighth, and in no case less than one-twelfth, of the span. The depth of trusses shall preferably be not less than one-sixth of the span. Panel lengths shall preferably be from 10 to 25 feet, and in truss bridges shall be so chosen that the angle between diagonal web members and the lower chord shall be not less than 50°.

16. Spacing of Stringers and Deck Girders.—Stringers shall be spaced 6 feet 6 inches center to center.

Deck girders less than 70 feet long shall be spaced 6 feet 6 inches center to center, deck girders over 70 feet long shall be spaced 6 inches farther apart for each 10 feet increase in length In bridges on curves, the center line between stringers, and between deck girders, shall be parallel to the chord of the curve between abutments, and at a distance from it equal to two-thirds the middle ordinate.

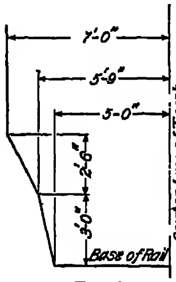


FIG 1

17. Half-Through Bridges.—Half-through truss bridges shall be avoided when possible. Where used, the flanges and brackets shall not come closer to the center line of track than shown in outline in Fig. 1.

18. Through Bridges.—In through bridges on straight track, no part of the structure shall come closer to the center line of the nearest track than shown in outline in Fig. 2. In bridges on curves, there shall be provided 1 inch additional clearance on each side of the track for each degree of curvature, and $2\frac{1}{2}$ inches additional clearance on the inside of the curve for each inch of superelevation of track.

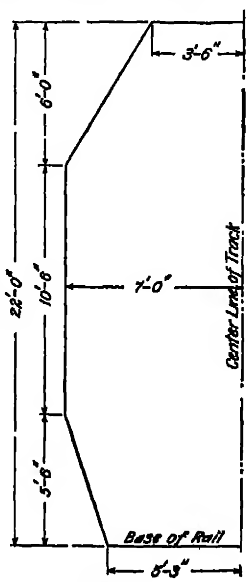


FIG. 2

19. Spacing and Gauge of Tracks.—Tracks shall be spaced 13 feet center to center, unless otherwise specified. The gauge of track is 4 feet $8\frac{1}{2}$ inches.

20. Spacing of Trusses.—The distance center to center of trusses shall preferably be not less than one-twelfth the span, and in no case less than one-half the depth of trusses.

21. Double-Track Spans.—Double-track spans shall preferably have only two trusses or girders. Where, on

account of thin floors, three-truss bridges are advisable, the tracks shall be spread so as to provide the proper clearance between the trusses for each track.

LOADING

22. Loads.—Bridges shall be designed to resist properly the stresses caused by the following forces. *dead load; live, or moving, load; impact and vibration, centrifugal force; wind pressure,* and *the longitudinal force due to suddenly stopping trains.*

23. Dead Load.—The dead load shall consist of the estimated weight of the entire structure. The weight of ties, guard timbers, and rails shall be taken as 400 pounds per linear foot of track, of timber as $4\frac{1}{2}$ pounds per board foot, and of ballast as 120 pounds per cubic foot. In truss bridges, two-thirds of the dead load shall, in general, be assumed as applied at the loaded chord, and one-third at the unloaded chord.

24. Live Load.—The live load on each track shall consist of two engines followed by a uniform load of 5,000 pounds per linear foot, as represented in Fig. 8, or a loading

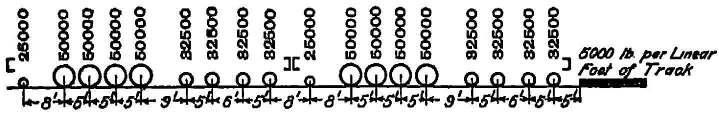


FIG. 8

having the same spacing of wheels and derived from the former by multiplying each load by the same number.

25. Impact and Vibration.—To provide for impact and vibration, an amount I is to be added to the stress or bending moment in each member, as given in the following formulas, in which

S = maximum live-load stress or bending moment in the member,

L = length, in feet, of single track that must be loaded in order to obtain the value S.

For counters, hip verticals, subverticals, short diagonals, floor members and connections, and members subject to reversal of stress,

$$I = S$$

For all other members,

$$I = \frac{300}{L + 300} S$$

26. Centrifugal Force.—In bridges on curves, the centrifugal force F shall be found from the formula

$$F = \frac{(4\,5 - 2D)}{100} D W$$

in which D = degree of curvature;
W = live load.

Centrifugal force shall be assumed to act 6 feet above the rail.

27. Wind Pressure.—Wind pressure shall be assumed as 300 pounds per linear foot on a train, applied 7 feet above the top of the rail, and 30 pounds per square foot on the exposed area of one girder in girder bridges, one truss in through bridges, or two trusses in deck bridges, together with the floor in truss bridges. When 50 pounds per square foot on twice the exposed area of one truss and the exposed area of the floor, with no train on the bridge, produces greater stresses than the above, the greater stresses shall be used.

28. Suddenly Stopping Trains.—The longitudinal force due to the friction between the rails and the wheels of suddenly stopping trains shall be taken as one-fifth of the live load on the structure.

DESIGN OF MEMBERS

29. Working Stresses.—All parts shall be so designed that the sum of the maximum stresses in any part shall not cause the intensity of stress to exceed the following values in pounds per square inch:

Tension on net section, 16,000.

Compression on gross section,

$$\frac{16,000}{1+\frac{l^2}{18,000\,r^2}}$$

in which l = unsupported length of member, in inches;
r = least radius of gyration, in inches.

In half-through truss bridges, the entire length of the upper chord shall be considered unsupported laterally.

Shear on net section of web-plates,

$$\frac{12,000}{1+\frac{d^2}{3,000\,t^2}}$$

in which t = thickness of web, in inches;
d = clear distance, in inches, between stiffeners or flange angles, whichever is the smaller.

The intensity of the shearing stress found by dividing the total vertical shear by the gross area of cross-section of the web shall in no case exceed 9,000.

Shear on shop rivets and pins, 11,000.

Shear on field rivets and turned bolts, 9,000.

Bending on pins, 22,000.

Bending on rolled sections and plate girders:

Tension, 16,000.

Compression, when unsupported length l of compression flange is not greater than twenty times the width w, 16,000.

Compression, when l is greater than $20\,w$,

$$20,000 - 200\frac{l}{w}$$

Bearing on shop rivets and pins, 22,000.

Bearing on field rivets, turned bolts, and ends of stiffeners, 18,000.

Bearing on masonry:

Sandstone and limestone, 300.

Cement concrete, 400.

Granite, 500.

Bearing on rollers shall not exceed $600\,D$ pounds per linear inch of roller, D being diameter of roller in inches.

30. Data.—The following general dimensions shall first be calculated or assumed:

Length of trusses, center to center of end pins or pedestals.

Length of girders, center to center of bearings.

Length of floorbeams, center to center of girders or trusses.

Length of stringers, center to center of floorbeams.

Depth of trusses and girders, center to center of gravity of chords or flanges.

31. Floor Members.—Solid floor sections, I beams, and channels shall be designed by their moments of inertia, or section moduli. The load on each axle shall be assumed to be distributed over a length of 3 feet in designing solid floor sections. In bridges on curves, stringers and deck girders shall be designed for the increase in load due to the eccentricity of the track.

32. Compression Members.—Pin and bolt holes shall be deducted from the gross section of compression members. The value of $\frac{l}{r}$ shall preferably be from 40 to 60, and must not exceed 100 for main members, nor 120 for members of lateral systems. Splices in compression members shall have sufficient rivets to fully develop the stresses in the members.

33. Tension Members.—The net section of a riveted tension member shall be determined by deducting from the gross section the area of cross-section of the greatest number of pin, bolt, or rivet holes that can be cut by a plane at right angles to the member. In addition, for rivets $\frac{7}{8}$ inch in diameter and larger, there shall be deducted each hole whose center lies within $\frac{3}{4}$ inch of the cutting plane, and a proportionate part of each hole whose center lies within $2\frac{3}{4}$ inches; and for rivets $\frac{3}{4}$ inch in diameter and smaller, each hole whose center lies within $\frac{1}{2}$ inch of the cutting plane, and a proportionate part of each hole whose center lies within 2 inches. Rivet holes shall be taken $\frac{1}{8}$ inch larger in diameter than the rivets.

34. Reversal of Stress.—Members subject to both tension and compression shall be designed to resist each stress plus eight-tenths of the other stress.

35. Combined Stresses.—Members subject to transverse stresses in addition to the direct stresses shall be designed for both.

36. Bearing Values of Rivets.—In calculating the bearing value of a rivet, the area subjected to stress shall be taken as equal to the product of the thickness of the plate and the diameter of the rivet before driving, that is, the nominal diameter. The value of countersunk rivets shall not be counted.

GENERAL DETAILS

37. General Requirements for Details and Connections.—Special attention shall be given to all details and connections; they shall always be of greater strength than the body of the member. All details shall be accessible for inspection, cleaning, and painting. Details that permit the collection of water shall be avoided if possible, if used, they shall be provided with drainage holes or filled with cement concrete.

38. Minimum Thickness.—No material less than $\frac{3}{8}$ inch thick shall be used except for latticing and fillers.

39. Single Angles.—Members, or sides of members, composed of single angles shall have both legs of each angle connected at the ends, or only 75 per cent. of the section shall be counted. No angle shall be smaller than 3 in. $\times$ 3 in. $\times$ $\frac{3}{8}$ in., nor be connected by less than four rivets, except for unimportant details.

40. Size of Rivets.—Rivets shall generally be $\frac{7}{8}$ inch and $\frac{3}{4}$ inch in diameter. The diameter of the rivet shall not be greater than one-fourth the width of the bar or angle through which the rivet passes, except for unimportant details, where $\frac{7}{8}$-inch rivets may be used in 3-inch angles, and $\frac{3}{4}$-inch rivets in $2\frac{1}{2}$-inch angles.

41. Spacing of Rivets.—Rivets $\frac{7}{8}$ inch in diameter shall be spaced not more than 6 nor less than 3 inches center to center, and placed not closer than $1\frac{3}{4}$ inches to any sheared edge nor closer than $1\frac{1}{2}$ inches to any rolled edge—except in special cases to conform to standards, where they may be placed not closer than $1\frac{1}{4}$ inches to a rolled edge. Rivets $\frac{3}{4}$ inch in diameter shall be spaced not more than 6 nor less than $2\frac{1}{2}$ inches center to center, and placed not closer than $1\frac{1}{2}$ inches to any sheared edge nor closer than $1\frac{1}{4}$ inches to any rolled edge—except to conform to standards, where they may be placed not closer than $1\frac{1}{8}$ inches to a rolled edge. The spacing of rivets at the ends of compression members shall not exceed four times the diameter of the rivets for a distance equal to the width of the member.

42. Grip of Rivets.—The grip of rivets shall preferably not exceed five times the diameter of the rivet, and shall in no case exceed 5 inches. When the grip is greater than 4 inches, the calculated number of rivets shall be increased 1 per cent. for each $\frac{1}{16}$ inch increase in grip.

43. Compression Members.—In compression members, the material shall mostly be concentrated at the sides. The unsupported widths of plates shall not exceed thirty times their thickness for web-plates, nor forty times their thickness for cover-plates of chords and end posts. No closed sections will be allowed.

44. Tie-Plates and Lattice Bars.—The open sides of all built-up members shall be stiffened by means of tie-plates and lattice bars. The length of tie-plates shall be not less than $1\frac{1}{4}$ times the width of the member. Double latticing shall preferably make an angle of about 45° with the axis of the member, and the bars shall be riveted where they cross each other. Single latticing shall preferably make an angle of about 60° with the axis of the member. Bars in single latticing shall have a thickness not less than one-fortieth, and in double latticing not less than one-sixtieth, of the length of the bar. Lattice bars shall be not less than $2\frac{1}{4}$ inches wide for members up to 9 inches, not less than

$2\frac{1}{2}$ inches for members from 9 to 15 inches, and not less than 3 inches for members more than 15 inches in width or depth.

45. Expansion.—Provision for expansion and contraction due to changes of temperature shall be made at the rate of 1 inch for every 100 feet.

46. Camber.—All trusses shall be cambered by giving the panels of the top chord an excess of length in the proportion of $\frac{1}{8}$ inch to every 10 feet. Plate girders shall not be cambered.

47. I Beams.—In short deck spans, when more than one I beam is used under a rail, the beams shall be bolted together with cast-iron separators between them, and connected by lateral bracing between the two sets of beams.

DETAILS OF FLOOR SYSTEMS

48. Ties, Guard Timbers, and Rails.—Ties, guard timbers, rails, wooden floors, and ballast, where necessary, will be provided and put in place by the railroad company. Cross-ties are 8 in. × 8 in., 10 feet long, framed to not less than $7\frac{1}{2}$ inches over bearings for stringers and girders 6 feet 6 inches center to center. The depth of the tie is increased 1 inch for each 6 inches additional width of girders. Ties are spaced 12 inches center to center, and every fourth tie is fastened to each stringer by a $\frac{3}{4}$-inch bolt. Guard timbers are 8 inches wide and 6 inches thick, framed to 4 inches over ties, and spaced 4 feet from inner edge to center of track. They are fastened by $\frac{5}{8}$-inch bolts to the ties that are connected to the stringers.

49. Floor Members.—Floor members shall be designed with special reference to stiffness, the depth of stringers shall be not less than one-eighth of the panel length, and that of floorbeams not less than one-sixth of the distance between trusses or girders.

50. Floor Connections.—Stringers shall be at right angles to the floorbeams, and shall be riveted to the floorbeam webs. If possible, they shall also rest on shelf angles

riveted to the webs of the floorbeams. Floorbeams shall preferably be at right angles to the girders or trusses. In half-through plate-girder bridges, the beams shall be riveted to the webs of the girders. In through truss bridges, the beams shall be riveted to the vertical posts, or to the web connection plates, if there are no vertical posts, diaphragms shall be riveted in, connecting the web connection plates at the ends of the floorbeams In deck truss bridges, the beams shall either be riveted to the trusses as in through bridges, or rest on the upper chords.

51. Connection Angles.—The connection angles of stringers to floorbeams, and floorbeams to girders and trusses, shall not be smaller than $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{9}{16}$ in. The fillers under connection angles shall be wider than the adjacent leg of the angle, and shall have two-thirds as many rivets through the projecting portion as through the portion under the angle.

52. Deck Bridges.—In deck girder bridges, the ties shall rest directly on the top flange. In deck truss bridges, the ties shall not rest directly on the top chord members; there shall always be a floor system with floorbeams at the panel points.

53. End Floorbeams.—All bridges with floor systems shall preferably be provided with end floorbeams of the same cross-section as intermediate floorbeams. Where this is impossible, the end stringers shall rest on the masonry; end struts, of nearly the same depth as the end stringers, shall be riveted to them and to the girders or trusses.

54. Solid Floors.—Solid floors shall preferably be made of a plate not less than $\frac{7}{16}$ inch thick riveted to the tops of longitudinal I beams supported by floorbeams.

DETAILS OF PLATE GIRDERS

55. Stiffeners.—Webs shall have stiffeners over bearing points, at points of local concentrated loadings, and at intervals not greater than the depth of the girder nor more

than 6 feet. Near the ends, the spacing of stiffeners shall be one-third to one-half the depth. They shall be composed of two angles, one on each side of the web, and shall fit tight between the horizontal legs of the flange angles For girders having flange angles with outstanding or horizontal legs 5 inches in width, stiffeners shall be not less than 4 in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in.; with outstanding legs 6 inches in width, not less than 5 in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in.; and with outstanding legs 8 inches in width, not less than 6 in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. Rivets in stiffeners shall be spaced not over $3\frac{1}{2}$ inches apart for a distance of 14 inches at each end, and not over 6 inches apart for the remaining distance. When the clear vertical distance between flange angles is less than fifty times the thickness of the web, stiffeners may be omitted—except over bearings, where they shall be designed to resist the reactions.

56. Web Splices.—Web-plates shall be spliced by one or more plates on each side; the splice plates shall have a section equal to at least three-fourths that of the web, and a pair of stiffeners shall be placed outside the plates. There shall be not less than two rows of rivets $3\frac{1}{2}$ inches apart on each side of the splice; rivets in splice plates shall be spaced not over $3\frac{1}{2}$ inches apart for a distance of 14 inches at top and bottom, and not over $4\frac{1}{2}$ inches between. Web splices shall be designed for the same resisting moment as the web.

57. Flange Rivets.—The pitch of rivets connecting the flange angles to the web at any point shall be calculated by the formula

$$p = \frac{K h_r}{V}$$

in which p = pitch, in inches;

K = smallest value of the rivet, in pounds;

h_r = vertical distance, in inches, between centers of rivet lines of flanges;

V = total maximum vertical shear, in pounds, at the section

When the ties rest on the top flanges of deck girders, the pitch of rivets in the top flange shall be 90 per cent. of the calculated pitch The rivets connecting flange plates to flange angles shall have the same spacing as, and stagger with, those connecting the flange angles to the web The spacing of rivets in plate-girder flanges shall in no case exceed $4\frac{1}{2}$ inches.

58. Flanges.—Flanges shall be designed by the net section of the bottom flange and by the gross section of the top flange One-eighth of the web shall be considered as part of the flange section. Girders with deep webs may have flanges composed of vertical as well as horizontal flange plates and secondary flange angles.

59. Flange Angles.—Flange angles shall preferably be of large sections; in general, not less than one-third to one-half the flange section shall be composed of angles.

60. Flange Plates.—Flange plates shall preferably have a thickness not greater than the angles, nor more than $\frac{3}{4}$ inch, when two or more plates are used, they shall have the same thickness, or shall diminish in thickness outwards from the angles, except the first plate in the top flange, which shall extend the full length of the flange, and may be thinner than the other plates Other plates shall extend 12 inches at each end beyond their theoretical ends. Flange plates shall extend beyond the outer lines of rivets not more than 4 inches, nor more than eight times the thickness of the thinnest plate.

61. Flange Splices.—Flange members of girders less than 70 feet long shall not be spliced. For girders longer than 70 feet, each flange angle shall be spliced by two splice angles each having a cross-section 75 per cent. that of the flange angle, and with sufficient rivets on each side of the splice to fully develop the stress in the splice angle. Flange plates shall preferably be spliced without additional splice plates, by continuing the outer plates beyond their theoretical ends a sufficient distance to splice the lower

plates. When splice plates are not in direct contact with the plates they splice, the calculated number of rivets shall be increased 20 per cent. for each intervening plate. Only one member shall be spliced at any section.

62. Riveting of Girders.—Deck girder bridges less than 70 feet long shall preferably be riveted up complete before shipping.

DETAILS OF RIVETED TRUSSES

63. Chord Members.—The chords and end posts shall be composed of channels, or of vertical plates and flange angles, connected by cover-plates at the top and by tie-plates and lattice bars at the bottom. Gusset plates for the connection of web members to chords shall be riveted to the inside of the chords, and shall be designed to resist the stresses to which they are subjected.

64. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and shall be thoroughly riveted to each other and to the connection plates at every intersection. Web members shall be composed of symmetrical sections, preferably not less than 12 inches in width, connected by web-plates or by tie-plates and lattice bars. The clear distance between gussets shall be not more than $\frac{1}{8}$ inch greater than the width of the web member that connects to them.

65. Connections and Splices.—Splices of chords shall be as close as practicable to panel points. All splices of chords and connections of web members shall have enough rivets to develop fully the stress in the members. If a splice occurs at a joint, that part of the gusset in contact with the chord shall be counted as a splice plate.

66. Tension Members.—Tension members shall be of the same general form as compression members. The use of flat bars alone for riveted tension members will not be allowed.

DETAILS OF PIN-CONNECTED TRUSSES

67. Chord Members.—The top chord and end posts shall be composed of channels, or of vertical plates with flange angles, connected by cover-plates at the top and by tie-plates and lattice bars at the bottom, or by tie-plates and lattice bars at both top and bottom. Splices of top chords shall be as close as practicable to panel points, and shall have enough rivets to develop the stresses fully. The bottom chord shall be composed of eyebars; the inside bars in the two end panels shall be connected to each other by diaphragms or by lattice bars. The eyebars shall be packed on the pins as narrow as possible; those in any panel shall not be in contact, and shall not diverge from the center line of truss by more than $\frac{1}{16}$ inch per foot.

68. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and pins shall be located at the intersections of these lines. Compression web members shall be composed of symmetrical sections, preferably not less than 12 inches in width, connected by web-plates or by tie-plates and lattice bars. Tension web members, except hip verticals and subverticals, shall be composed of eyebars. Hip verticals and subverticals shall be of the same general form as compression members.

69. Counters.—Counters shall be adjustable eyebars with screw ends and open turnbuckles. The area at the root of an upset screw end shall in no case be less than 10 per cent. greater than the body of the bar. No counter shall have a sectional area of less than 3 square inches.

70. Minimum Eyebars.—No eyebar shall be less than 4 inches in width or less than $\frac{3}{4}$ inch in thickness.

71. Pins.—Pins shall be not less than 3 inches in diameter, and shall project $\frac{1}{4}$ inch at each end beyond the outside surfaces of the members.

72. Riveted Tension Members.—Riveted tension members shall have a net section back of pinholes at least

equal to the net section of the member, and through pinholes at least 25 per cent greater.

73. Pin Plates.—Where necessary for section or bearing, members shall be reinforced at pinholes by pin plates Each plate shall contain sufficient rivets to transmit its proportion of the bearing pressure to the member. One plate on each side shall extend at least 6 inches beyond the end of the tie-plate. The cross-section of a compression member through a pinhole shall be at least equal to that of the member.

DETAILS OF STEEL TRESTLES (VIADUCTS)

74. Towers and Main Spans.—Steel trestles shall consist of riveted spans on trestle bents braced in pairs to form towers. Tower spans shall be not less than 30 feet long, and shall be riveted to the tops of the trestle bents; main spans shall be riveted to the tops of the trestle bents at one end, and bolted to them at the other through expansion holes. In single-track trestles, the girders shall be connected to the tops of the columns; in double-track trestles, the outer lines of girders or trusses shall be connected to the tops of the columns, and the inner lines to cross-girders, the ends of which are connected to the tops of the columns.

75. Trestle Bents.—Trestle bents for single-track trestles shall be not less than 8 feet wide on top, and the batter of each post shall be not less than 1 horizontal to 6 vertical. Trestle bents for double-track trestles shall be not less than 19 feet 6 inches wide on top, and the batter of each post shall be not less than 1 horizontal to 8 vertical On curves, towers shall be placed so that the center lines of the bents are at right angles to the chord of the curve between bents.

76. Towers.—Towers shall be divided into stories not more than 30 feet in height by horizontal struts and diagonal bracing between the columns.

77. Negative Reactions.—In estimating negative (downward) reactions at the feet of the columns, the weight of train shall be taken as 800 pounds per linear foot.

DETAILS OF BEARINGS

78. Bedplates.—The ends of all spans and the bottoms of columns of trestle bents shall rest on bedplates or pedestals, and shall be held in place by anchor bolts. Bedplates for girders and stringers shall be not less than 1 inch in thickness, and for trusses and columns not less than $1\frac{1}{2}$ inches. Holes for anchor bolts may be $\frac{1}{4}$ inch larger in diameter than the bolts

79. Anchor Bolts.—Anchor bolts shall be not less than 1 inch in diameter for girders and stringers, nor less than $1\frac{1}{4}$ inches for trusses; they shall be set in holes drilled in the masonry, and the holes shall be filled with cement grout Anchor bolts for columns having a negative reaction shall be designed to resist the reaction, and shall be built in a mass of masonry the weight of which is not less than twice the estimated reaction. Anchor bolts in expansion ends shall be so placed that the ends can move freely in the direction of expansion, and in no other direction.

80. Pedestals.—Spans over 75 feet in length shall have pin bearings and pedestals at both ends. Pedestals shall be built up of base and web plates not less than $\frac{3}{4}$ inch in thickness. The webs shall be secured to the base plates by angles not less than 6 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in., with the 6-inch leg vertical, and the webs shall be connected to each other. The pedestals shall be of sufficient height to distribute the load over the bearings.

81. Ends of Columns.—Caps and base plates shall be connected to the tops and bottoms, respectively, of all viaduct columns, by means of horizontal angles not less than 6 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in., with the 6-inch leg vertical or parallel to the batter of the column.

82. Rollers.—Spans over 75 feet in length shall have rollers at one end. Rollers shall be not less than 3 inches in diameter, and shall be turned down to a groove $\frac{1}{4}$ inch deep to fit guiding strips of this thickness on the bearing

plates above and below the rollers. Special attention shall be given to roller bearings, so that they will not hold water, and so that they can be readily cleaned.

83. Adjacent Spans.—When the girders or trusses of two adjacent spans rest on the same pier, the bedplates and pedestals shall be entirely independent for each girder or truss.

84. Spans on Grade.—For spans without rocker bearings, a sole plate of the same size as the bedplate shall be riveted to the bottom of the span at each end; if the track is on a grade, the sole plate shall be planed to bevel, so that the lower surface will be level when the floor of the span is parallel to the grade.

DETAILS OF BRACING

85. Independent Bracing.—All spans shall be independently braced; no bracing shall be used in common for any two adjacent spans.

86. Style of Members.—Members of bracing shall either be built-up members or be composed of simple rolled shapes. They shall intersect each other, and the members to which they connect, on lines passing as nearly as practicable through their centers of gravity, and shall be riveted to each other and to connection plates at every intersection No member shall be less than $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in., and no connection shall have less than four rivets.

87. Lateral Bracing.—Top and bottom lateral bracing shall be provided in deck and through bridges; bottom lateral bracing in half-through bridges Lateral bracing shall be riveted to the stringers of the floor system wherever it comes in contact with them Deck girders shall have the top lateral bracing so arranged that the length of the flange between lateral connections will not exceed twelve times its width. If stringers are longer than twelve times the width of the flange, lateral bracing shall be riveted to their upper flanges.

88. Transverse Bracing.—Deck girder bridges shall have transverse frames, of the same depth as the girders,

riveted to the stiffeners near the ends, and at other points at distances apart not greater than 15 feet If stringers are longer than twenty times the width of the flange, transverse frames shall be riveted to their webs at the intersections of the stringers and lateral bracing. Deck truss bridges shall have sway-bracing, of the same depth as the trusses, at every panel point.

89. Knee Bracing.—Half-through bridges shall have brackets or knee braces riveted to the floorbeams or the tops of solid floors, and to the webs of the girders. Knee braces shall fit tight under the top flange angles of girders, and shall be as wide at the top of the rail as the clearance will allow. They shall be so arranged that the distance between them shall not exceed twelve times the width of the top flange of the girder.

90. Portal Bracing.—Through bridges shall have portals and portal brackets, and intermediate brackets at each transverse strut of the upper lateral bracing. Portals shall be as deep as the specified clearance will allow. Where the headroom above the track is 25 feet or more, sway frames shall be provided at every panel point of the top chord; they shall be as deep as the required headroom will allow.

DESIGN OF HIGHWAY AND STREET-RAILWAY BRIDGES

GENERAL DIMENSIONS

91. Kinds of Bridges.—The following kinds of bridges shall preferably be used:

For spans less than 35 feet in length, rolled beams.

For spans from 35 to 100 feet in length, plate girders.

For spans from 100 to 150 feet in length, riveted trusses.

For spans longer than 150 feet, pin-connected trusses.

If, for any reason, it is desired to depart more than 10 feet from these limits, permission in writing must be obtained from the Engineer.

92. Panel Lengths and Depths.—The depth of girders shall preferably be not less than one-twelfth the span. Panel lengths shall preferably be from 15 to 30 feet, and in truss bridges the panel lengths and depths shall be so chosen that the inclined web members shall make an angle with the lower chord of not less than 50°. The depth of I beams shall in no case be less than one-thirtieth of the span.

93. Spacing of Stringers, Girders, and Trusses. For bridges carrying only a railway track, stringers of floor systems, and deck girders less than 70 feet long, shall be spaced 6 feet 6 inches center to center. Deck girders over 70 feet long shall be spaced 6 inches farther apart for each 10 feet increase in length. Stringers, girders, and trusses in bridges carrying both railways and highways, or highways only, shall be arranged to accommodate the actual traffic, and shall be adapted to local conditions. Trusses shall be spaced not less than one-twentieth of the span.

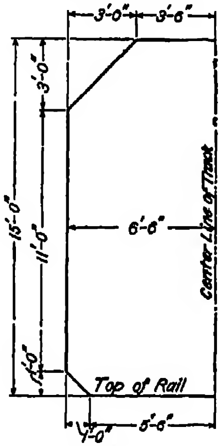


FIG 4

94. Clearance.—No part of any bridge shall come closer to the center line of the nearest track than is shown in outline in Fig. 4. If a track is on a curve, ½ inch additional clearance for each degree of curvature shall be provided on the outside of the curve, and on the inside of the curve ¼ inch additional clearance for each degree of curvature, and 2 inches for each inch of superelevation of track. All through highway bridges carrying railways shall have a clear headroom of 15 feet at a distance of 3 feet from the wheel-guards; those carrying highways only shall have a clear headroom of not less than 13 feet at a distance of 3 feet from the wheel-guards.

95. Spacing of Tracks.—When there is more than one track, the tracks shall be assumed as 10 feet center to center.

LOADING

96. Loads.—Bridges carrying highways only shall be designed to resist properly the stresses caused by the following forces. *dead load*, *live* or *moving load*, and *wind pressure*. Bridges carrying railways alone or in connection with highways shall be designed to resist properly the stresses caused by the forces mentioned and, in addition, those caused by *impact* and *vibration*, *centrifugal force*, and *the longitudinal force due to suddenly stopping cars.*

97. Dead Load.—The dead load shall consist of the estimated weight of the entire structure. The actual weight of the floor and track, if any, shall be computed; timber shall be assumed to weigh 4½ pounds per board foot. For bridges carrying railways alone, the weight of rails, ties, etc. may be taken as 300 pounds per linear foot of track. In truss bridges, one-half the weight of the trusses shall be assumed as applied at the loaded chord, and one-half at the unloaded chord; the entire weight of floor shall be assumed as applied at the loaded chord.

98. Live Load.—The live load shall consist of the estimated maximum moving loads that the bridge is expected to carry. It will depend on the amount and kind of traffic to which the bridge is to be subjected, and shall in general be assumed as follows:

1. *For City Bridges Subject to Heavy Loads.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members of all spans, either a uniform load of 100 pounds per square foot on all parts of the floor, or a steam road roller weighing 20 tons distributed as represented in Fig. 5. For the girders or trusses of all spans up to 100 feet, a uniform load of 100 pounds per square foot on the entire surface of the floor; of all spans over 200 feet, 80 pounds per square foot, and of intermediate spans, proportional intermediate values (Between 100 and 200 feet, the uniform load decreases 1 pound per square foot for each 5 feet increase in span.)

2. *For Bridges in the Suburbs of Cities and in Well-Settled Town Districts.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members of all spans, either a uniform load of 100 pounds per square foot on all parts of the floor, or a steam road roller weighing 15 tons distributed as represented in Fig. 6. For the girders or trusses of all spans up to 100 feet, a uniform load of 80 pounds per square foot on the entire surface of the

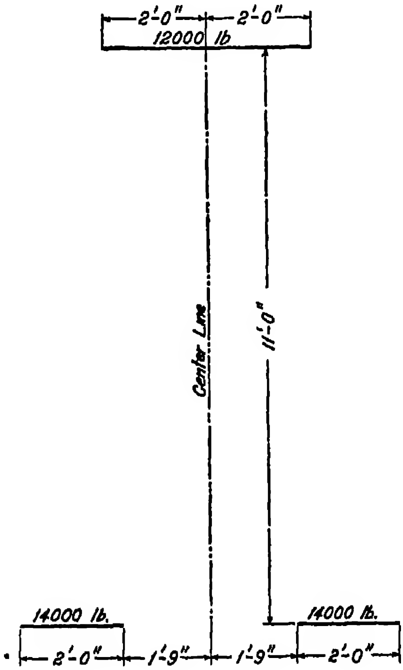


FIG. 5

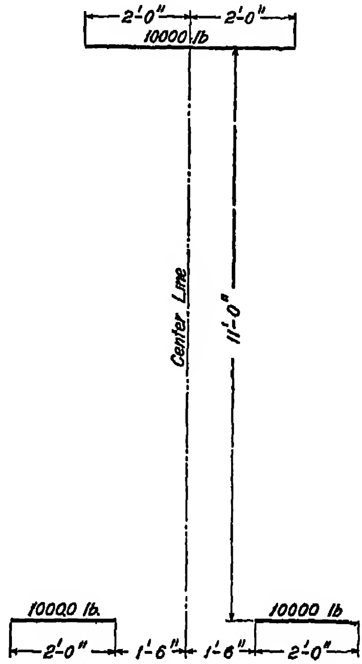


FIG. 6

floor; of all spans over 200 feet, 60 pounds per square foot; and of intermediate spans, proportional intermediate values. (Between 100 and 200 feet the uniform load decreases 1 pound per square foot for each 5 feet increase in span.)

3. *For Bridges in Country Districts and in Thinly Settled Communities.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members and connections

of all spans, either a uniform load of 80 pounds per square foot on all parts of the floor, or a steam road roller weighing 15 tons distributed as represented in Fig. 6. For the girders or trusses of all spans up to 75 feet, a uniform load of 80 pounds per square foot on the entire surface of the floor, of all spans over 200 feet, 55 pounds per square foot, and of intermediate spans, proportional intermediate values. (Between 75 and 200 feet, the uniform load decreases 1 pound per square foot for each 5 feet increase in span.)

4. *For All Bridges Carrying Street Railway, or That Are Expected to Carry Street Railway in the Near Future*.—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members of all spans, and for the girders of spans less than 75 feet, an electric car on each track weighing 40 tons, distributed as shown in Fig 7; for the web members of spans 75 to 100 feet in length, 1,600 pounds per linear foot, and of all spans over 100 feet, a floorbeam load of $(1,600\,p)$ pounds for each track at $\left(\frac{100}{p}\right)$ floorbeams (p being the panel length, in feet); for the chord members of spans 75 feet in length, 1,600 pounds per linear foot; of spans 275 feet or more in length, 1,000 pounds per linear foot; and of intermediate spans, proportional intermediate values. (Between 75 and 275 feet, the uniform load decreases 3 pounds for each 1 foot.)

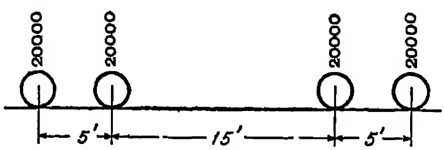


FIG 7

5. *For Bridges Carrying Both Highway and Street Railway*.—The live load shall consist of the electric car or uniform load given in item 4 above, together with the uniform loads given in items 1, 2, or 3 covering the entire floor, except a width of 10 feet for each track.

99. Impact and Vibration.—To provide for impact and vibration in bridges carrying street railways, an amount I is to be added to the stress or bending moment caused by the car in each member, as given by the following formulas:

For counters, hip verticals, subverticals, short diagonals, floor hangers, floor members and connections, members subject to reversal of stress, and all other members for which L is less than 25 feet,

$$I = \tfrac{3}{10} S$$

For members for which L is greater than 25 feet and less than 200 feet,

$$I = \frac{300 - L}{1,000} S$$

For all members for which L is greater than 200 feet,

$$I = \tfrac{1}{10} S$$

Here, S = maximum live-load stress or bending moment in the member due to load on car track;
L = length of track, in feet, that must be loaded in order to obtain the value S.

100. Wind Pressure.—Wind pressure shall be assumed in girder bridges as 50 pounds per square foot on the exposed area of one girder, and in truss bridges as 50 pounds per square foot on twice the exposed area of one truss together with the exposed area of the floor. In designing the stringers of floor systems in bridges carrying railway, the wind pressure on the cars shall be assumed to be 250 pounds per linear foot, applied 6 feet above the rail, and the center of moments for the increase in load on the leeward stringer shall be taken at the top of the rail.

101. Centrifugal Force.—In bridges on curves, the centrifugal force F caused by electric cars shall be assumed to act 5 feet above the rail, and shall be found by the formula

$$F = \left(\frac{1.5 - .05\,D}{100}\right) D\,W$$

in which D = degree of curve;
W = live load.

102. Suddenly Stopping Cars.—The longitudinal force due to a suddenly stopping car shall be taken equal to 16,000 pounds applied at the top of the rail.

DESIGN OF MEMBERS

103. Working Stresses.—All parts shall be so designed that the sum of the maximum stresses shall not cause the intensities of stress to exceed the following values, in pounds per square inch:

Tension on net section, 16,000.

Compression on gross section,

$$\frac{16{,}000}{1+\frac{l^2}{18{,}000\,r^2}}$$

in which l = unsupported length of member, in inches;
r = least radius of gyration, in inches.

In half-through truss bridges, the entire length of the upper chord shall be considered unsupported laterally, the stiffening effect of knee braces being ignored.

Shear on net sections of web plates,

$$\frac{12{,}000}{1+\frac{d^2}{3{,}000\,t^2}}$$

in which t = thickness of the web, in inches;
d = clear distance, in inches, between stiffeners or flange angles, whichever is the smaller.

The intensity of shearing stress found by dividing the total vertical shear by the gross area of cross-section of the web shall in no case exceed 9,000.

Shear on shop rivets and pins, 11,000.

Shear on field rivets and turned bolts, 9,000.

Bending on pins, 22,000.

Bending on rolled sections and plate girders:

Tension, 16,000.

Compression, when unsupported length l of compression flange is not greater than twenty times the width w, 16,000.

Compression, when l is greater than 20 w,

$$20{,}000 - 200\,\frac{l}{w}$$

Bearing on shop rivets and pins, 22,000.

Bearing on field rivets, turned bolts, and ends of stiffeners, 18,000.

Bearing on masonry:

Sandstone and limestone, 300.

Cement concrete, 400.

Granite, 500.

The bearing on rollers shall not exceed 600 D pounds per linear inch of roller, in which D is the diameter of roller, in inches.

Bending on fir, yellow-pine, and white-oak beams, 1,200.

Bending on white-pine and spruce beams, 1,000.

104. Data.—The following general dimensions shall first be calculated or assumed:

Length of trusses, center to center of end pins or pedestals.

Length of girders, center to center of bearings.

Length of floorbeams, center to center of girders or trusses.

Length of stringers, center to center of floorbeams.

Depth of trusses and girders, center to center of gravity of chords or flanges.

105. Compression Members.—Pin and bolt holes shall be deducted from the gross section of compression members. The value of $\frac{l}{r}$ will preferably be from 40 to 60, and must not exceed 100 for main members, nor 120 for members of lateral systems. Splices in compression members shall have sufficient rivets to develop fully the stresses in the members.

106. Tension Members.—The net section of a riveted tension member shall be determined by deducting from the gross section the area of cross-section of the greatest number of pin, bolt, or rivet holes that can be cut by a plane at right angles to the member. In addition, for rivets $\frac{7}{8}$ inch in diameter and larger, there shall be deducted each hole whose center lies within $\frac{7}{8}$ inch of the cutting plane and a proportional part of each hole whose center lies within $2\frac{3}{4}$ inches; and, for rivets $\frac{3}{4}$ inch in diameter and smaller,

there shall be deducted each hole whose center lies within ½ inch and a proportional part of each hole whose center lies within 2 inches. Rivet holes shall be taken ⅛ inch larger in diameter than the rivets.

107. Reversal of Stress.—Members subject to both tension and compression shall be designed to resist each stress plus eight-tenths of the other stress.

108. Combined Stresses.—Members subject to transverse stresses in addition to the direct stresses shall be designed for both kinds of stress.

109. Bearing Values of Rivets.—In calculating the bearing value of a rivet, the area subjected to stress shall be taken equal to the product of the thickness of the plate and the diameter of the rivet before driving, that is, the nominal diameter. The value of countersunk rivets shall not be counted.

110. Floor Stringers.—In bridges with steel stringers, each stringer shall be designed to support one-half the load on a front wheel of a steam roller and one-half the load on one rear roller, forming a system of two concentrated loads. In bridges with wooden joists, the loads on each roller of a steam road roller may be assumed to be distributed over a width 12 inches greater than the width of the roller, and the portion that goes to each stringer or joist calculated on this basis.

GENERAL DETAILS

111. Details and Connections.—Special attention shall be given to all details and connections, which shall always be of greater strength than the body of the member. All details shall be accessible for inspection, cleaning, and painting. Details that permit the collection of water shall be avoided if possible; if used, they shall be provided with drainage holes or filled with cement concrete.

112. Minimum Thickness.—No material less than $\frac{5}{16}$ inch thick shall be used except for latticing and fillers,

and for webs of channels, which may be $\frac{1}{4}$ inch thick. If the bridge is over a steam railroad, no material less than $\frac{3}{8}$ inch thick shall be used below the floor, except for buckled plates, for which $\frac{5}{16}$-inch material may be used under sidewalks.

113. Single Angles.—Members or sides of members composed of single angles shall have both legs of each angle connected at the ends, or only 75 per cent. of the section shall be counted. No angle shall be smaller than $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in., nor be connected by less than three rivets, except for unimportant details.

114. Sizes of Rivets.—Rivets shall generally be either $\frac{7}{8}$ inch or $\frac{3}{4}$ inch in diameter. The diameter of the rivet shall not be greater than one-quarter the width of the bar or angle through which it passes, except for unimportant details, where $\frac{7}{8}$-inch rivets may be used in 3-inch angles, and $\frac{3}{4}$-inch rivets in $2\frac{1}{2}$-inch angles.

115. Spacing of Rivets.—The distance center to center of rivet holes shall be not less than three times the diameter of the rivets, nor more than 6 inches in any line. The centers of rivet holes shall not be closer to the edge of any piece than $1\frac{1}{2}$ times the diameter of the rivet. The spacing of rivets at the ends of compression members shall not exceed four times the diameter of the rivet for a distance equal to the width of the member. For the remainder of the length of compression members, the distance center to center of rivets shall be not greater than sixteen times the thickness of the thinnest outside plate.

116. Compression Members.—In compression members, the material shall be concentrated at the sides as much as possible. The unsupported width of plates shall not exceed thirty times their thickness for web-plates, nor forty times for cover-plates of chords and end posts. No closed sections will be allowed.

117. Tie-Plates and Lattice Bars.—The open sides of all built-up members shall be stiffened by means of tie-plates and lattice bars. The length of tie-plates shall be not

less than their width. Double latticing shall preferably make an angle of about 45° with the axis of the member, and the bars shall be riveted where they cross each other. Single latticing shall preferably make an angle of about 60° with the axis of the member The thickness of tie-plates shall be not less than one-fortieth the distance between their rivet lines; the thickness of bars in single latticing shall be not less than one-fortieth, and in double latticing one-sixtieth of the distance between the rivets in their ends. Lattice bars shall be not less than $2\frac{1}{4}$ inches wide for members up to 9 inches, not less than $2\frac{1}{2}$ inches for members from 9 to 15 inches, nor less than 3 inches for members more than 15 inches in width or depth.

118. Expansion.—Provision for expansion and contraction shall be made at the rate of 1 inch for every 100 feet of span

119. Camber.—All trusses shall be cambered by giving the panels of the top chord an excess of length in the proportion of $\frac{1}{8}$ inch for each 10 feet. Plate girders shall not be cambered.

DETAILS OF FLOOR SYSTEMS

120. Wooden Floors.—In bridges constructed solely for street-railway purposes, ties shall be of yellow pine not less than 5 inches wide, 7 inches deep, and 10 feet long, and shall be framed to $6\frac{1}{2}$ inches over bearings for stringers and girders 6 feet 6 inches center to center. For girders over 6 feet 6 inches center to center, the depth of tie shall be increased $\frac{1}{2}$ inch for each 6-inch increase in the spacing of the girders. Ties shall be spaced not more than 12 inches center to center, and every fourth tie shall be fastened at both ends to the stringers or girders by $\frac{3}{4}$-inch bolts. Guard timbers of yellow pine not less than 6 in $\times$ 6 in., framed to 4 inches over ties, shall be bolted by $\frac{5}{8}$-inch bolts to the ties that are bolted to the stringers, and so that the inner edge of the guard timber shall be 4 feet from the center of the track Guard timbers shall extend over all piers and abutments.

Superelevation of rails on curves shall be provided for as may be required in each case.

121. In bridges with wooden floors constructed for combined street-railway and highway purposes, the rails shall preferably be supported on yellow-pine ties not less than 6 in. × 6 in. in size, nor less than 8 feet long, spaced not over 15 inches center to center and resting on the stringers. Longitudinal nailing pieces not over 2 feet apart shall be spiked to the top of the tie; they shall be not less than 3 inches wide, and of sufficient height to bring the top of the plank floor level with the top of the rails. A plank floor not less than 2 inches thick, preferably of spruce or oak, shall be nailed to the nailing pieces.

122. For bridges in country districts and in thinly settled communities, the highway portion of the floor shall preferably consist of one layer of white-oak plank not less than 3 inches in thickness laid at right angles to the trusses or girders, and with joints about $\frac{1}{4}$ inch open. Wooden stringers not less than 3 in. × 12 in. or steel beams with wooden nailing pieces shall be used, and the former shall be spaced not over 2 feet 6 inches center to center. In general, the width of wooden stringers shall be not less than one-fourth of the depth. The plank floor shall be securely spiked to the stringers.

123. For bridges in the suburbs of cities, in well-settled town districts, and in some cases for city bridges not subject to heavy loads, the highway portion of the floor shall preferably consist of two layers of plank; the lower layer shall be of white oak not less than 3 inches in thickness, and shall be laid diagonally with joints not over $\frac{1}{4}$ inch open; the upper layer shall be of white oak or spruce 2 inches thick, laid tight at right angles to the girders or trusses and securely spiked to the lower layer. Wooden stringers, as before, may be used, but steel stringers with wooden nailing pieces shall have the preference. When one layer of floor plank is used, the distance, in feet, between the centers of joists or nailing pieces shall not be greater than the thickness of the plank, in inches. When more than one layer is used, the clear distance,

in feet, between joists or nailing pieces shall not be greater than the thickness of the lower layer, in inches.

124. Paved Floors.—For city bridges subject to heavy loads, the floor shall preferably consist of prepared wooden blocks, asphalt, brick, or granite blocks. Wooden blocks shall be given the preference; granite blocks shall be used only in the immediate vicinity of warehouses, docks, or freight houses, where the traffic is exceedingly heavy and continuous. Paved floors shall be supported on buckled plates securely riveted to the upper flanges of stringers and to the floorbeams. Buckled plates shall be laid with the buckle hanging down, and shall be covered with cement concrete having a thickness not less than 3 inches under the roadway nor less than 2 inches under the sidewalk. Between the concrete and the paving there shall be spread a cushion coat of clean, sharp sand, perfectly free from moisture, to an even thickness of 1 inch. Open joints between blocks shall be filled with cement grout or coal-tar pitch.

125. Wheel-Guards.—In bridges with wooden floors, a wheel-guard of timber not less than 6 inches wide and 4 inches thick shall be placed longitudinally on the floor. The upper edge of the wheel-guard shall be 6 inches from the surface of the floor. The edge of the guard toward the roadway shall be 6 inches from the clearance line of the trusses or girders. In bridges with paved floors, a metal or stone wheel-guard shall be provided, it shall be 6 inches high and at least 6 inches outside of the clearance line of the trusses or girders.

126. Floor Connections.—Stringers shall be at right angles to the floorbeams, and shall be riveted to the floorbeam webs If possible, the stringers shall also rest on shelf angles riveted to the webs of the floorbeams. Floorbeams shall preferably be riveted to the webs of the girders in half-through girder bridges, and to the vertical posts or web connection plates in through truss bridges; in deck truss bridges, the floorbeams shall either be riveted to the trusses, as in through bridges, or rest on the top chords. Where sidewalks are supported outside the girders or trusses, the

floorbeams shall be extended under the sidewalks, or sidewalk brackets shall be riveted to the outsides of the posts, and their top flanges connected to those of the floorbeams.

The connection angles of stringers to floorbeams and of floorbeams to girders or trusses shall be not less than 3 in. × 3 in × $\frac{7}{16}$ in The fillers under connection angles shall be twice as wide as the adjacent leg of the angle, and shall have two-thirds as many rivets through the projecting portion as through the portion under the angle.

127. Hand Railing.—A suitable hand railing or fence shall be placed at each side of the bridge. The railing shall be not less than 3 feet 6 inches above the top of the floor, and have not more than 6 inches clearance beneath it.

DETAILS OF PLATE GIRDERS

128. Stiffeners.—Webs shall have stiffeners over bearing points, at points of local concentrated loadings, and at intervals not greater than the depth of the girder nor more than 6 feet. Near the ends, the spacing of stiffeners shall be one-third to one-half the depth. Stiffeners shall be composed of two angles, one on each side of the web, and shall fit tight between the horizontal legs of the flange angles. For girders having flange angles with outstanding or horizontal legs 5 inches in width, stiffeners shall be not less than $3\frac{1}{2}$ in × $3\frac{1}{2}$ in × $\frac{5}{16}$ in.; with outstanding legs 6 inches in width, not less than 4 in. × $3\frac{1}{2}$ in. × $\frac{5}{16}$ in ; with outstanding legs 8 inches in width, not less than 5 in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. Rivets in stiffeners shall be spaced not over $4\frac{1}{2}$ inches apart for a distance of 18 inches at each end, and not over 6 inches apart for the remaining portion. When the clear vertical distance between flange angles is less than fifty times the thickness of the web, stiffener angles may be omitted, except over bearings. Stiffeners over bearings shall be designed to resist the reactions.

129. Web Splices.—Web-plates shall be spliced by one or more plates on each side, the plates on each side to have

a section equal to three-fourths that of the web, and a pair of stiffeners placed outside the plates. There shall be not less than two rows of rivets $3\frac{1}{2}$ inches apart on each side of the splice. Rivets in splice plates shall be spaced not over $3\frac{1}{2}$ inches apart for a distance of 14 inches at top and bottom, and not over 5 inches between. Web splices shall be designed for the same resisting moment as the web.

130. Flange Rivets.—The pitch of rivets connecting the flange angles to the web at any point shall be calculated by means of the formula $p = \frac{K h_r}{V}$ (see Art. **57**). When the ties of street-railway bridges rest on the top flanges of deck girders, the pitch of the rivets in the top flange shall be 90 per cent. of the calculated pitch. The rivets connecting flange plates to flange angles shall have the same spacing as, and stagger with, those connecting the flange angles to the web The spacing of rivets in plate-girder flanges shall in no case exceed 6 inches.

131. Flanges.—Flanges shall be designed for the net section of the bottom flange, and the gross section of the top flange. One-eighth of the web shall be considered as part of the flange section. Girders with deep webs may have flanges composed of vertical as well as horizontal flange plates.

132. Flange Angles.—Flange angles shall preferably be of large sections, in general, not less than one-third to one-half the flange section shall be composed of angles.

133. Flange Plates.—Flange plates shall preferably have a thickness not greater than the angles nor more than $\frac{3}{4}$ inch. When two or more plates are used, they shall have the same thickness, or shall diminish in thickness outwards from the angles, except the first plate of the top flange, which shall extend the full length of the flange and may be thinner than the other plates Other plates shall extend 12 inches at each end beyond their theoretical ends. Flange plates shall extend beyond the outer lines of rivets not more than 4 inches nor more than eight times the thickness of the thinnest plate.

134. Flange Splices.—Flange members of girders less than 70 feet long shall not be spliced. For girders longer than 70 feet, each flange angle shall be spliced by two splice angles, each having a cross-section 75 per cent. of that of the flange angle, and with sufficient rivets on each side of the splice to develop fully the stress in the splice angle. Flange plates shall preferably be spliced without additional splice plates, by continuing the outer plates beyond their theoretical ends a sufficient distance to splice the lower plates. When splice plates are not in direct contact with the plates they splice, the calculated number of rivets shall be increased 20 per cent. for each intervening plate. Only one member shall be spliced at any section.

DETAILS OF RIVETED TRUSSES

135. Chord Members.—The chords and end posts shall be composed of channels, or of vertical plates and flange angles, connected by cover-plates at the top, and by tie-plates and lattice bars at the bottom. Gusset plates for the connection of web members to chords shall be riveted to the inside of the chords, and shall be designed to resist the forces to which they are subjected.

136. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and shall be thoroughly riveted to each other and to the connection plates at every intersection. Web members shall be composed of symmetrical sections, connected by web-plates or tie-plates and lattice bars. The clear distance between gussets shall be not more than $\frac{1}{8}$ inch greater than the width of the web members that connect to them.

137. Connections and Splices.—Splices of chords shall be as close as practicable to panel points. All splices of chords and connections of web members shall have enough rivets to develop fully the strength of the members. If a splice occurs at a joint, that part of the gusset in contact with the chord shall be counted as a splice plate.

138. Tension Members.—Tension members shall be of the same general form as compression members. The use of flat bars alone for riveted tension members will not be allowed.

DETAILS OF PIN-CONNECTED TRUSSES

139. Chord Members.—The top chord and end posts shall be composed of channels, or of vertical plates with flange angles, connected by cover-plates at the top, and by tie-plates and lattice bars at the bottom, or by tie-plates and lattice bars at both top and bottom Splices of top chords shall be as close as practicable to panel points, and shall have enough rivets to develop fully the stresses in the members. The bottom chord shall be composed of eyebars; the inside bars in the two end panels shall be connected to each other by diaphragms or by lattice bars. The eyebars shall be packed on the pins as narrow as possible, those in any panel shall not be in contact, and shall not diverge from the center line by more than $\frac{1}{16}$ inch per foot.

140. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and pins shall be located at the intersections of these lines. Compression web members shall be composed of symmetrical sections, connected by web-plates or by tie-plates and lattice bars. Tension web members, except hip verticals and subverticals, shall be composed of eyebars. Hip verticals and subverticals shall be of the same general form as compression members.

141. Counters.—Counters shall be adjustable eyebars with screw ends and open turnbuckles. The area at the root of an upset screw end shall in no case be less than 10 per cent. greater than the cross-sectional area of the body of the bar. No counter shall have a sectional area of less than 2 square inches.

142. Pins.—Pins shall be not less than $2\frac{1}{2}$ inches in diameter, and shall project $\frac{1}{4}$ inch at each end beyond the outside surfaces of the members.

143. Riveted Tension Members.—Riveted tension members shall have a net section back of the pinholes equal to that of the member, and through the pinholes 25 per cent greater.

144. Pin Plates.—Where necessary for section or bearing, members shall be reinforced at pinholes by pin-plates. Each plate shall contain sufficient rivets to transmit its proportion of the bearing pressure to the members; one plate on each side shall extend at least 6 inches beyond the end of the tie-plate The cross-section of a compression member through the pinhole shall be equal to that of the member.

DETAILS OF STEEL TRESTLES

145. Tower and Main Spans.—Steel trestles shall consist of riveted spans on trestle bents, braced in pairs to form towers. Tower spans shall be not less than 30 feet long, and shall be riveted to the tops of the trestle bents. Main spans shall be riveted to the tops of the trestle bents at one end, and bolted to them at the other through expansion holes. Girders and trusses may be connected to the tops of the columns or to cross-girders that are connected to the tops of the columns.

146. Trestle Bents.—The batter of columns in trestle bents shall be not less than 1 horizontal to 8 vertical. In trestles for single-track railway only, bents shall be not less than 8 feet wide on top, and if the width is less than 10 feet, the batter of the columns shall be not less than 1 horizontal to 6 vertical. On curves, towers shall be placed so that the center lines of the bents are at right angles to the chord of the curve between the bents.

147. Towers.—Towers shall be divided into stories not more than 30 feet in height, by horizontal struts and diagonal bracing between the columns.

148. Negative (Downward) Reactions.—In estimating negative (downward) reactions at the feet of the

columns, wind pressure shall be assumed to have an intensity of 50 pounds per square foot, acting on an area equal to twice the exposed area of the unloaded trestle.

DETAILS OF BEARINGS

149. Bedplates.—The ends of all spans, and the bottoms of columns of trestle bents, shall rest on bedplates or pedestals, and shall be held in place by anchor bolts. Bedplates for girders and stringers shall be not less than $\frac{3}{4}$ inch in thickness; and for trusses and columns, not less than 1 inch. Holes for anchor bolts may be $\frac{1}{4}$ inch larger in diameter than the bolts.

150. Anchor Bolts.—Anchor bolts shall be not less than 1 inch in diameter; they shall be set in holes drilled in the masonry, and the holes shall be filled with cement grout. Anchor bolts for columns having negative reactions shall be designed to resist the reaction, and shall be built in a mass of masonry the weight of which is not less than twice the estimated reaction. Anchor bolts in expansion ends shall be so placed that the ends can move freely in the direction of expansion and in no other direction.

151. Pedestals.—Spans over 75 feet in length shall have pin bearings and pedestals at both ends. Pedestals shall preferably be built up of base plates and web-plates, not less than $\frac{5}{8}$ inch in thickness. The webs shall be secured to the base plates by angles not less than 5 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in., with the 5-inch leg vertical, and the webs shall be connected to each other. Pedestals shall be of sufficient height to distribute the load over the bearings.

152. Ends of Columns.—Cap and base plates shall be connected to the tops and bottoms, respectively, of all trestle columns by means of horizontal angles not less than 5 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in., with the 5-inch leg vertical or parallel to the batter of the column.

153. Rollers.—Spans over 75 feet in length shall have rollers at one end. Rollers shall be not less than 3 inches

in diameter, and shall be turned down to a groove $\frac{1}{4}$ inch deep to fit guiding strips of this thickness on the bearing plates above and below the rollers. Special attention shall be given to roller bearings, so that they will not hold water and so that they can be readily cleaned.

DETAILS OF BRACING

154. All spans shall be independently braced; no bracing shall be used in common for any two adjacent spans.

155. Style of Members.—Members of bracing shall be composed of simple rolled sections or built-up members. No member shall be less than $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in., and no connection shall have less than three rivets.

156. Lateral Bracing.—Top and bottom lateral bracing shall be provided in all deck and through bridges, except deck plate girders less than 5 feet deep, where the bottom bracing may be omitted Bottom lateral bracing shall be provided in half-through bridges. Lateral bracing shall be riveted to the stringers of the floor system wherever they come in contact with them. Deck girders shall have the top lateral bracing so arranged that the length of flange between lateral connections will not exceed twenty times the width of flange

157. Transverse Bracing.—Deck girder bridges shall have transverse frames of the same depth as the girders riveted to the stiffeners near the ends and at other points at distances apart not greater than 20 feet. Deck truss bridges shall have at every panel point sway-bracing of the same depth as the trusses.

158. Knee Bracing.—Half-through bridges shall have brackets or knee braces riveted to the floorbeams and to the webs of the girders. The brackets shall fit tight under the flange angles of the girders, and extend out to within 3 inches of the edge of the wheel-guard.

159. Portal Bracing.—Through bridges shall have portals and portal brackets, and intermediate brackets at

each transverse strut of the upper lateral bracing. Portals shall be as deep as the specified headroom will allow. Where the headroom above the floor is 20 feet or more, sway frames shall be provided at every panel point of the top chord. They shall be as deep as the required headroom will allow.

SPECIFICATIONS FOR THE CONSTRUCTION OF STEEL BRIDGES

MATERIALS

CHEMICAL AND PHYSICAL PROPERTIES

160. Kind of Material.—The materials used in the construction of bridges shall generally be rolled steel, steel castings, and wrought iron. In case other materials are required, additional specifications will be furnished relating to them.

161. Grades of Steel.—Steel shall be made only by the open-hearth process. Rolled steel shall be of two grades; namely, *rivet steel* and *structural steel.* Rivet steel shall be used for rivets; structural steel shall be used for all other purposes, unless steel castings or wrought iron are especially called for.

162. Rivet Steel.—Rivet steel shall contain not more than .04 per cent. of phosphorus, nor more than .04 per cent. of sulphur. It shall have an ultimate tensile strength of not less than 46,000, nor more than 54,000, pounds per square inch, an elastic limit of not less than 60 per cent of the ultimate strength, an elongation of not less than 28 per cent., including the break, and a reduction of area of not less than 55 per cent. Rivet rods shall bend double, cold, one side flat on the other, without cracking of the outer fibers.

163. Structural Steel.—Structural steel shall contain not more than 08 per cent. of phosphorus for acid steel, nor more than .04 for basic steel, and not more than .04 per

cent. of sulphur for either kind of steel. It shall have an ultimate tensile strength of not less than 56,000, nor more than 64,000, pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation of not less than 28 per cent., and a reduction of area at fracture of not less than 50 per cent. The fracture shall appear fine-grained, silky, and bluish gray, and shall be entirely free from hard and granular spots.

Test pieces shall bend double, cold, until the surfaces touch each other, without cracking of the outer fibers. They shall stand punching and cold reaming to 1½ times the diameter of the punched hole without cracking the edges of the hole. Angles of all thicknesses shall, while cold, open flat, and if under ⅝ inch thick shall bend close shut without showing signs of fracture.

164. Steel Castings.—Steel castings shall contain not more than 08 per cent. of phosphorus for acid steel, nor more than .05 per cent. for basic steel, nor more than 05 per cent of sulphur for either kind of steel. They shall have an ultimate tensile strength of not less than 60,000 pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation of not less than 18 per cent., and a reduction of area at fracture of not less than 25 per cent. Steel castings shall be fine-grained, homogeneous, and free from blowholes and other defects. A test piece 1 in. × ½ in. shall bend cold through an angle of 90° around a rod whose diameter is 1½ inches, without showing signs of fracture.

165. Wrought Iron.—Wrought iron shall be, as nearly as practicable, the best grade of pure iron. It shall be double-rolled, fibrous, tough, uniform in character, and thoroughly welded in rolling. It shall be entirely free from surface defects. It shall have an ultimate tensile strength of not less than 50,000 pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation, including the break, of not less than 18 per cent., and a reduction of area at fracture of not less than

25 per cent. Test pieces shall bend, when cold, through an angle of 180° around a rod whose diameter is twice the thickness of the test piece, without showing signs of fracture.

MILL TESTS

166. Reports of Tests and Melt Numbers.—The Contractor shall furnish the Engineer, free of charge, a report showing the physical and chemical properties of every melt that goes into the material of the bridge. The chemical analysis shall show the amount of carbon, phosphorus, sulphur, and manganese contained in each melt. The melt numbers shall be clearly stamped on all finished material, and omission to do so, or changes, or confusion of the melt numbers may be cause for rejection.

167. Test Pieces.—Two test pieces shall be cut from the finished material of every melt. Test pieces of rivet steel shall be round rods having the same diameter as the rivets. Test pieces of structural steel for pins and rollers shall be round rods turned to the form and size shown in Fig. 8. The elongation shall be measured in a length of 2 inches, and include the break.

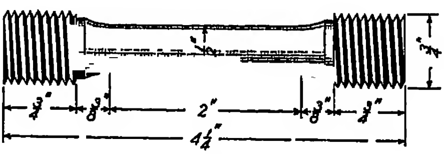


FIG 8

Test pieces of structural steel for other purposes, and of wrought iron, shall be flat bars of the same thickness as the material from which the test pieces are cut, and shall have the form and size shown in Fig. 9. The elongation shall be measured in a length of 8 inches, including the break.

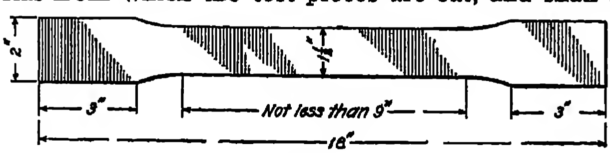


FIG 9

Test pieces of steel castings shall be cast with one or more castings, and shall be cut from them when they are

cold; they shall be turned to the form and size shown in Fig. 8, and the elongation shall be measured in a length of 2 inches, including the break.

168. Annealed Material.—Two test pieces shall be cut from material that is to be annealed, one before and the other after annealing.

169. Varying Sections.—Test pieces shall be cut from the thickest and from the thinnest materials when sections differing by $\frac{3}{8}$ inch or more are rolled from the same melt, and from each section when widely different sections, such as angles and I beams, are rolled from the same melt.

170. Labor and Tools.—The Contractor shall furnish the Engineer, free of charge, proper test pieces, machines, tools, and labor necessary to make the required tests.

171. Rejection of Materials.—If material does not possess the specified properties, the entire melt from which the material was taken shall be rejected, unless by additional tests it can be proved that the defects are confined to only a part of the melt. All material that, subsequently to its acceptance at the mills, shows that it is not of the desired quality shall be rejected and shall be replaced with satisfactory material. Rusty material shall be rejected, unless it is thoroughly freed from rust before being used.

FULL-SIZE TESTS

172. Full-sized members shall be tested to destruction if so directed by the Engineer. The material in such members shall be paid for by the Engineer at the contract price per pound, if it complies with the requirements, otherwise, it will be rejected at the Contractor's expense, and all similar members may be rejected unless it can be shown, by additional tests, that the failure is due to defects that are confined to one or a very few members.

173. Eyebar Tests.—In general, one full-size test shall be made from every twenty-five eyebars. After being

properly annealed, eyebars shall have an ultimate tensile strength of not less than 55,000 pounds per square inch, an elastic limit of not less than 50 per cent. of the ultimate strength, and an elongation of not less than 15 per cent. in 10 feet, including the break. If a bar breaks in the head, but develops the required ultimate strength and elongation, it shall not on that account be rejected, unless more than one-third of the number of bars tested break in the head.

WORKMANSHIP

GENERAL REQUIREMENTS

174. Finished Material.—Workmanship and finish shall be first-class and in accordance with the best practice. Material shall get a thorough rolling at the proper temperature; large sections shall be rolled from large-sized billets; pins shall be forged in the most approved manner. Finished material shall be entirely free from surface defects, shall have good finish, and be well straightened in the mill before shipment.

175. Variation in Dimensions.—Material shall be rolled as near as practicable to the weight and thickness specified, no variation will be allowed greater than $2\frac{1}{2}$ per cent. above nor $1\frac{1}{2}$ per cent. below the computed weights, except in wide-sheared plates, for which slightly greater variation will be allowed, according to the practice in the best rolling mills.

176. Annealing.—All parts that have been heated during manufacture shall be carefully annealed and thoroughly cooled before they are prepared for connections.

177. Welding.—Welds in steel will not be allowed under any circumstances. Welds in wrought iron will be permitted when specified.

178. Reentrant Angles.—No sharp reentrant angles will be allowed in any piece of metal; the corners shall always be drilled out before the sides are cut.

179. Rivet Holes.--Rivet holes in members of longitudinal and lateral bracing, stiffeners, and unimportant details may be punched not more than $\frac{1}{16}$ inch larger than the nominal diameter of the rivets. Rivet holes in flanges, the edges of web-plates where flanges are attached, floor connections, and all riveted members of trusses and their connections shall be punched $\frac{3}{16}$ inch smaller, and reamed to not more than $\frac{1}{16}$ inch larger than the nominal diameter of the rivets, if the material is $\frac{11}{16}$ inch thick or less. Reaming shall preferably be done after the parts are assembled. Rivet holes in material $\frac{3}{4}$ inch thick and over, and in flanges of I beams and channels, shall be drilled from the solid metal.

180. Punching.—Rivet holes shall be spaced and punched so accurately that, when parts are brought together, the corresponding holes will match. Slight inaccuracies may be corrected by reamers. Drifting to make holes large enough for the rivets will not be allowed in the shop or in the field.

181. Reaming and Drilling.—When parts are assembled for reaming, at least one-third of the holes shall be filled with bolts. If necessary to take them apart for shipment, they shall be match-marked, and a diagram of the marks shall be furnished the erector to insure the same position of each part in the finished bridge as in the shop. Parts assembled for drilling shall be taken apart, and any shavings between them removed before riveting. Burrs on reamed and drilled work shall be removed

182. Field Connections.—All field connections, except for members of bracing, shall be fitted in the shop. The rivet holes shall be reamed to fit while the members are bolted together in their correct positions, or by means of metal templets not less than $1\frac{1}{4}$ inches thick, carefully clamped to the members in the correct position.

183. Rivets.—The sizes of rivets shown on the plans shall be taken to mean the diameters of the cold rivets before driving. When heated and ready for driving, the surfaces of all rivets shall be perfectly clean; when driven, they

shall completely fill the holes and be perfectly tight. Loose and badly driven rivets shall be cut out and replaced with tight well-driven rivets. Rivet heads shall be round and of uniform size for the same-sized rivets all through the work; they shall be full and neatly made, concentric with the shank, and in full contact with the surface.

184. Riveting.—Wherever possible, rivets shall be driven by machine or power riveters. Power riveters shall be capable of maintaining the applied pressure after driving. Field riveting shall preferably be done by pneumatic riveting hammers. Before any rivets are driven, at least one-third the holes shall be filled with bolts of the same size as the rivets, and they shall be carefully tightened.

185. Bolts.—Bolts shall not be used in place of rivets, except by special permission. When bolts are used, the holes shall be exactly at right angles to the surfaces of the connected parts, and the bolts shall be turned to a driving fit.

186. Riveted Members.—After being riveted, members shall be straight and correct in dimensions. Great care shall be taken that the bearing surfaces of girders and the faces of flange angles of girders and riveted truss members are perfectly straight.

187. Web-Plates.—Web-plates shall be straight and not project beyond the faces of the flange angles; they shall be not more than $\frac{1}{4}$ inch below the faces of the angles at any point. Splices in web-plates shall be not more than $\frac{1}{8}$ inch open.

188. Flange Members.—Where flange members of plate girders are spliced, in either the top or the bottom flange, the ends shall be planed exactly square; and, after riveting, the spliced ends shall be in perfect contact throughout the entire section of the spliced member.

189. Stiffeners.—Stiffeners shall fit tight between the horizontal legs of flange angles; the ends of fillers under stiffeners, and of web splice plates shall be not more than $\frac{1}{8}$ inch from the edges of the vertical legs of the flange angles.

190. Ends of Floor Members.—The ends of floor-beams and stringers shall be planed perfectly smooth and straight; after planing, the members shall have the length shown on the plans. Not more than $\frac{1}{16}$ inch shall be planed off the faces of the connection angles. Ends of solid floor sections shall be perfectly straight and smooth; if necessary, they shall be planed to secure this result.

191. Ends of Riveted Members.—Where riveted members, either tension or compression, are spliced, the ends shall be planed smooth exactly at right angles to the axis of the member; and, after riveting, the spliced ends shall be in perfect contact throughout the entire section of the spliced member. The ends of columns of viaducts shall be planed smooth before the cap and base plates are riveted on, and so that the entire section of the column, as well as the faces of the horizontal angles riveted to the ends, shall have a full and even bearing on the cap and base plates.

192. Ends of Girders.—The ends of all girders shall be neatly finished; web-plates, flange angles, and flange plates shall be finished flush with each other.

193. Eyebars.—Eyebars shall be of uniform thickness throughout, perfectly straight, and free from welds. The heads shall be full, smooth, and sound, and accurately centered with the bars; they shall be formed by upsetting in the most approved manner. After the heads are formed, eyebars shall be carefully annealed and thoroughly cooled before further handling.

194. Pinholes. Pinholes shall be bored exactly at right angles to the axis of the member, not more than $\frac{1}{50}$ inch larger in diameter than the pins up to 5 inches diameter, nor more than $\frac{1}{32}$ inch larger for diameters greater than 5 inches, and not more than $\frac{1}{64}$ inch greater or less than the calculated distance center to center as shown on the drawings. The centers of pinholes in riveted members shall generally lie on a line passing through the center of gravity of the member, unless shown elsewhere on the drawings

The centers of pinholes in eyebars shall be on the center line. Bars that are to be placed side by side in a bridge shall be bored so accurately that, if stacked up one above the other, a pin of the required size can be passed simultaneously through all the holes at either end without much forcing.

195. Pins.—Pins shall be forged and carefully turned cylindrical, smooth, and true to size, and long enough to give all members a full bearing. They shall be driven with pilot nuts and caps; at least one driving cap and pilot nut for each size of pin shall be furnished by the Contractor. Threads on ends of pins shall project $\frac{1}{4}$ inch beyond the surfaces of the nuts when they are screwed on.

196. Pin Nuts.—Pin nuts shall be made so as to enclose the projecting ends of pins and come to a full bearing against the members.

197. Rollers.—Rollers shall be forged and carefully turned cylindrical, smooth, and true to size.

198. Bearings.—Sole plates and bedplates shall be planed smooth and straight. The sliding surfaces at expansion ends shall be planed in the direction of expansion. The bottoms of webs and connection angles of pedestals shall be planed before the base plates are riveted on.

199. Steel Castings.—Steel castings shall be planed where noted on drawings and wherever else it is necessary to insure good workmanship and even bearing. Cored holes shall be not more than $\frac{1}{8}$ inch greater or less than the required sizes, nor more than $\frac{1}{8}$ inch from the position shown on the drawings. Steel castings shall be true to the required dimensions after annealing.

200. Shipment.—All pins, rivets, and other small parts shall be boxed, and the screw threads wrapped with twine, before shipment. An excess of field rivets equal to 20 per cent. of the required number for each size and length shall be shipped for each bridge. All members shall be handled and loaded on cars in such a way as to avoid injury; any piece

showing the effects of rough handling may be rejected. The weight and erection mark shall be plainly marked on each part, and the weight and contents on each box.

PAINTING

201. General.—As soon as material is finished anc accepted, it shall be thoroughly cleared of rust, dirt, scale and other surface deposits, and carefully painted in accord ance with the following specifications. No painting shal be done until material is accepted.

202. Surfaces in Contact.—Surfaces that will be ir contact with others shall be given one coat of red lead anc linseed oil before assembling.

203. Inaccessible Parts.—All parts not accessible for painting after erection shall be given one heavy coat o approved paint at the shop as soon as finished and accepted and one coat at the bridge site before erection.

204. Machined Surfaces.—All machined surfaces, sucl as screw threads, pins, and bearing surfaces shall be coatec with a mixture of white lead and tallow as soon as finishec and before leaving the shop.

205. Finished Members.—Finished members shall b given one heavy coat of approved paint before leaving th shop. In general, paint shall be allowed to dry 48 hour before loading material for shipment.

206. Painting After Erection.—After erection, th bridge shall be given two heavy coats of approved paint. A least 48 hours must elapse between the applications of th two successive coats to any part of the bridge.

207. Weather Conditions.—Painting shall be don only when the surface of the metal is perfectly dry. Fiel painting shall not be done in wet or freezing weather. Sho painting may be done in such weather, if, after painting, th material is allowed to remain at least 48 hours in a covere building whose inside temperature is not below freezing.

208. Quality.—The quality of paint and class of labor for painting shall be the best obtainable; special attention shall be given to this part of the work.

ERECTION

209. Commencement of Work.—The Contractor shall notify the Engineer when he is ready to commence work, and the erection shall not begin until authority has been received in writing from the Engineer

210. Care of Material.—Before and during erection, all material shall be kept clean and so stored and handled as to avoid injury.

211. Old Structures.—If the new bridge is to take the place of an old bridge on the same site, the Contractor shall take down the old bridge; if required, he shall take it down without loss or injury to any part, and shall mark all parts for reerection. A diagram showing these marks shall be furnished to the Engineer.

212. Method of Erection.—If it is necessary to place any restrictions on the method of erection, the Engineer shall state them in the letter of invitation to bid; he shall also state the desired disposal of the old bridge.

213. Lines and Grades.—The Contractor will be expected to preserve with care all stakes set by the Engineer.

214. Field Riveting in Splices.—Field rivets in splices of compression members shall not be driven until the members are subjected to dead-load stress. The splices shall be well bolted prior to this, to hold the members firmly in line.

215. Bridge Seats.—Bridge seats shall be dressed by the Contractor. If they are out of level, he shall place the bedplates or pedestals level and at the correct elevation; if necessary, he shall fill in under them with cement grout well rammed into all open spaces under the bedplates or pedestals. The grout shall be allowed to set at least 24 hours before any load is placed on the bedplate or pedestal.

216. Laws and Ordinances.—The Contractor shall comply with all laws and ordinances applying to and governing the work of erection, and shall obtain all necessary permits and comply with their requirements. He shall take precautions to guard against accidents and injury to persons and property, and shall be responsible for all losses due to floods, storms, and other casualties. He shall so conduct his work as not to interfere with the work of other Contractors, nor with the traffic on railroads, highways, or waterways, unless he procures written permission to do otherwise.

217. Extra Work.—If the Engineer erects the bridge and extra work is found to be necessary owing to defective shop work or careless handling, the Contractor shall bear the cost of it; this cost shall be deducted from the amount due him.

218. Employment of Men.—The Contractor shall follow all reasonable directions of the Engineer in regard to the discipline of his men during the work of erection. At the completion of the work he shall, if desired by the Engineer, furnish proper bond to protect the Engineer from all liabilities resulting from the failure of the Contractor to pay for the materials or labor.

219. Final Test.—As soon as the bridge is completed and before its final acceptance, the Engineer may test it by loading it with the specified loads Any defect that becomes apparent shall be corrected by the Contractor.

220. Name Plate.—A name plate of neat design and finish, giving the name of the Contractor and the date of erection, shall be firmly attached to each bridge in a prominent position.

INSPECTION

221. Inspectors.—All material shall be tested by, and all workmanship shall be under the supervision of, inspectors appointed by the Engineer. The Engineer and his inspectors shall have free access at all times to all parts of the mills and shops in which any part of the bridge is being manufactured.

222. Reports of Inspectors.—Inspectors shall report the results of all tests, they shall report the shipments of material from the mills to the shops, and check the shipments off from the bills of material as fast as they are made. They shall also report the progress of the work, and in their final report shall state if the work as a whole was carried out in a satisfactory manner, noting any errors that may have been made.

223. Mill Orders and Shipping Invoices.—When the Contractor places the orders for the material, he shall at the same time inform the Engineer as to the order numbers, the furnace where the ingots are cast, and the mills where the material is rolled He shall also send two complete copies of the mill orders, and shall arrange to have the inspectors furnished with complete copies of shipping invoices with each shipment.

224. Facilities for Testing.—The Contractor shall furnish, free of charge, all facilities, labor, tools, and instruments or machines necessary for inspection, testing, and weighing.

GENERAL REMARKS

225. Engineering Work.—All the engineering work in connection with the design and construction of bridges is done by one or more bridge engineers and their assistants in the employ of the city, town, or company that is to build the bridge. In some cases, engineers are employed permanently, such as city engineers and chief and bridge engineers of railroad companies; in other cases, they are employed temporarily, and only for the purpose of designing and superintending the construction of one or more bridges.

226. Letter of Invitation.—Before the design of the bridge is begun, certain dimensions and conditions must be known These can be tabulated for almost all bridges in the form given below, the blanks being filled out for each bridge, and a copy furnished the designer. In some cases

GENERAL DATA

For bridge over____________________________

at____________________________________

Length and general dimensions__________________

____

Skew or angle of abutments with center line of bridge____

____

Width of bridge and location of trusses____________

____

Floor system______________________________

Number and location of tracks__________________

Loading__________________________________

Description of abutments_____________________

Distance from floor to clearance line_____________

" " " " high water________________

" " " " low water_________________

" " " " river bottom_______________

Character of river bottom____________________

Usual season for floods_____________________

Name of nearest railroad station________________

Distance to nearest railroad station______________

Time limit_______________________________

Name of Engineer__________________________

Address of Engineer________________________

Remarks_________________________________

____

____

a copy is sent to the bidder with the letter of invitation, and he is requested to submit a proposed plan with his bid. In the majority of cases, however, the bidder is furnished with a plan that gives all the information called for on the blank form, and in addition the stresses in the members.

227. When it is necessary to have a bridge ready for traffic at a certain date, this date is stated in the letter of invitation. In some cases, bidders are offered a bonus for each day the bridge is finished before, and required to pay a penalty for each day the bridge is finished after, the stated time.

228. Location.—The first step in the design of a bridge is the selection of a site, and the arrangement of spans and location of piers and abutments. No fixed rule can be given for the location of the bridge and abutments or piers; these are matters that depend almost wholly on local conditions, and must be decided by the judgment of the engineer. In general, however, it may be said that the site should be chosen with regard to economy and so that the bridge will be in a good position to accommodate the traffic. As a rule, single spans are most economical for short bridges, and steel trestles for long bridges. If the bridge is over a river, and a steel trestle cannot be used, owing to the danger from floods or because the piers would block navigation, the bridge may be composed of two or more spans resting on masonry piers; or, if at a great elevation above the river, so that high piers are expensive and objectionable, a single long span may be used. It is frequently necessary to estimate the approximate cost of several plans and designs before one is finally selected.

229. Kind of Bridge.—When the location of the piers has been decided, the number and length of spans may be determined; then the general style of bridge, whether deck or through, the kind of trusses or girders, together with panel lengths and depths, and the width of bridge, are selected. In general, deck bridges should be used when possible, as they are somewhat more economical and, on

account of the transverse bracing between the trusses, somewhat stiffer than through bridges; their use is limited, however, to locations where there is plenty of room below the floor. The kind of structure—plate girders, riveted or pin-connected trusses, etc.—depends on the span; the style of truss—Pratt, Warren, Baltimore, etc.—is left to the judgment of the engineer. The depth and panel length are, as a rule, controlled by the specifications; but a few words may be added. It has been found in practice that for through plate-girder bridges, panel lengths of 10 to 15 feet are best; for riveted truss bridges, from 15 to 20 feet; and for pin-connected truss bridges, from 20 to 30 feet. Panels may be made longer than 30 feet in spans greater than 250 feet in length, but for shorter spans it is better not to exceed this limit.

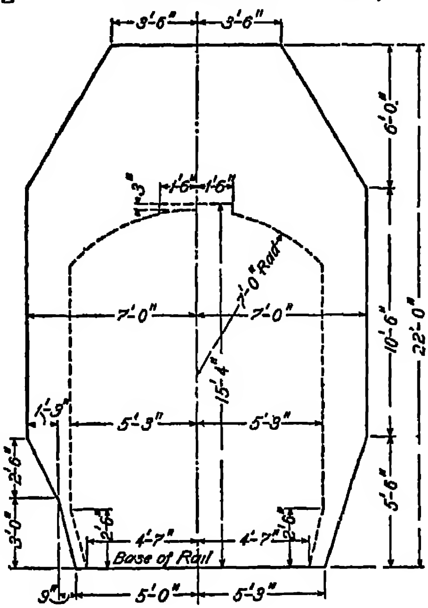


Fig. 10

230. Width and Clear Height.—The width and clear height must be sufficient to accommodate the traffic. For railroad bridges, they depend on the outside dimensions of the car or engine having the largest cross-section; or, since one car may have the greatest width and another the greatest height, the outside dimensions of a car that would have that width and height. This is usually referred to as the **maximum equipment**. The dotted lines in Fig. 10 represent the approximate outline of the maximum equipment in use on steam railroads in the United States, the full lines forming the diagrams given in Figs. 1 and 2. The latter are

commonly spoken of as **clearance diagrams.** Their lines are located somewhat outside the lines of the maximum equipment to give required clearance, so as to keep unlooked-for projections, such as heads and arms out of car windows, from striking any part of the bridge. The additional height at the top is to prevent any part of the overhead bracing from striking the heads of brakemen on top of the cars. Owing to difficulties of design, the top flanges of through plate girders are allowed to come closer to the lower part of the outline of maximum equipment than any part of truss bridges. The same clearance diagram should be used for bridges on railroads that are undergoing electrification, that is, on which the motive power is being changed from steam to electricity, as well as on roads that are built especially for heavy cars operated by electricity on private right of way for freight and passenger service. The indications at present are that the equipment on such roads will be as large as the largest now in use on steam railroads.

As the equipment on street railroads is somewhat smaller than on steam or heavy electric roads, less width and height need be provided. Fig. 4 represents the outline of one-half the clearance diagram used on street railways. Less clearance is needed above the cars than on steam roads, as it is not necessary to allow for men standing on top of the cars, but simply to provide ample room for the trolley.

On both steam and electric roads, more clearance is allowed at the sides on curves than on straight track, as parts of the cars overhang the track and the tops lean over.

231. The width of a highway bridge depends altogether on the amount and kind of traffic to be provided for. For country bridges, the clear width should never be less than about 16 feet; for suburban and city bridges, the width may be anything from about 20 feet to the full width of the street leading to the bridge. Each case must be decided by the local requirements.

232. Floor System.—The arrangement of stringers and floorbeams depends, to a great extent, on the allowable

depth of floor from the surface to the underneath clearance line. As a rule, it is well to have as much depth as possible. Two-truss bridges are preferred, as a center truss requires the spreading of the tracks or roadways. When two-truss bridges are used, the depth of the floorbeams and that of the floor are much greater than for three-truss bridges. In the elimination of grade crossings, and frequently in other cases, it is exceedingly expensive to separate the grades enough to permit the use of deep floorbeams; and it is then advisable at times to spread the track on railway bridges, or separate the roadways on highway bridges, so as to allow for the insertion of a center truss. In such cases, the floorbeams need not be so deep, as they will be but one-half as long as if two trusses were used.

Some engineers think it is best to place the stringers of railroad bridges directly under the rails, but it seems better practice to space them 6 feet 6 inches center to center, each one about 9 inches from the center of the rail. In this way, some of the shock and vibration of the train is absorbed by the elasticity of the ties, which have a chance to deflect.

Two stringers are frequently placed close side by side to carry the load on each rail, but this is not good practice, as it is almost impossible to distribute the load equally between them on account of poor fitting of ties, etc. If this inequality occurs at the center of a panel, the stringer getting the greater part of the load will deflect, thereby transmitting some of the load to the other stringer, and not much harm will result. If the inequality occurs at the end of a panel, the rivets connecting one of the stringers to the floorbeam web are liable to get the load that should go through two sets of rivets; they will be overstrained, and may, in time, loosen. When stringers are riveted to floorbeam webs, it is better to use one stringer for each rail.

233. Short-span I-beam bridges for railroads are composed of two or three beams for each rail. The beams are placed close together, and are made to act as one beam by being firmly bolted together and held in place by separators.

There is then no objection to using more than one beam under a rail. If the stringers in floor systems were laid close and bolted together in the same way, it would be impossible to get satisfactory connections to the floorbeams.

234. Deck truss railroad bridges are occasionally built with a floor system, the ties resting on the top chords of the

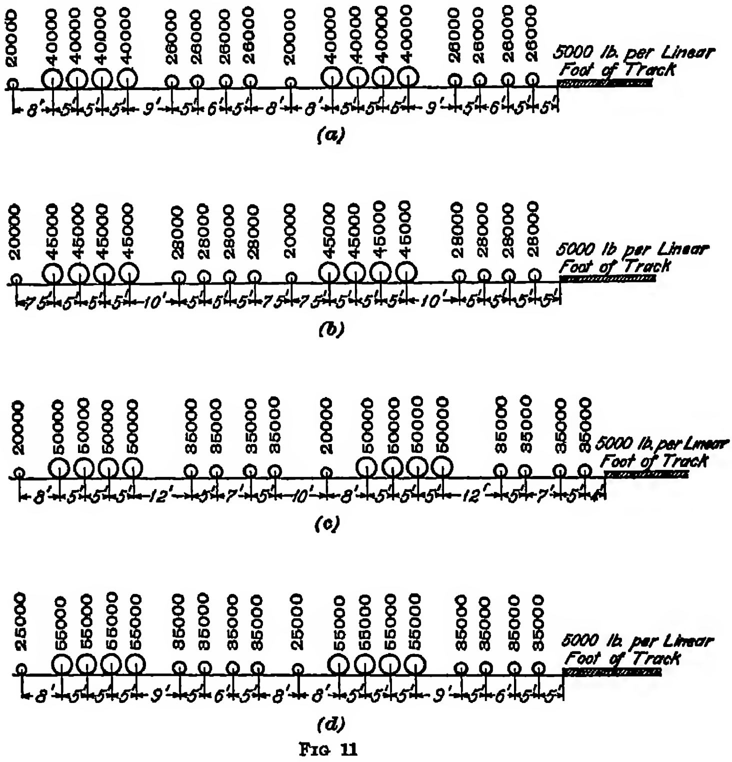


FIG 11

trusses. The sections of top chord then act as beams as well as compression members, and are subjected to simultaneous compressive and bending stresses. This practice is condemned by the best engineers. It is best in all cases to provide a floor system in the top chord, the ends of the floorbeams being connected to the insides of the trusses or resting on top of the trusses at the panel points.

235. Live Loads.—One of the most important steps in the design of a bridge is to ascertain the live or moving load the bridge is to carry. The live load on a railroad bridge consists of locomotives and cars. As explained in *Stresses in Bridge Trusses*, Part 4, it is customary to use typical loadings that represent the heaviest loads it is expected the bridge will ever have to carry. Fig. 11 shows four typical loadings in use on leading railroads in the United States; Cooper's loadings also have been adopted by many of the leading railroad companies. At the present time, Cooper's E50 well represents the heaviest loads on most American railroads. In a few special cases, where the loads in use are somewhat heavier than this, each load of Cooper's E50 may be multiplied by 1.1 or 1.2, as desired, giving what may be called E55 and E60, respectively, approximately equivalent to the actual loads, and the resultant systems substituted for E50 in the specifications. For bridges on branch lines and on lines on which the locomotives and cars are light, E40 will give sufficiently heavy loads. In case extra heavy locomotives and cars are used for any purpose, the bridges on lines over which they operate should be designed for the actual loads, increased 10 per cent. in some cases to allow for future increase.

236. Up to the present time, the only type of concentrated load that has been considered for railroad bridges has been the steam locomotive and train of cars. There are now in use electric locomotives very nearly equal in weight to the steam locomotives. Fig. 12 shows the distances between axles and the weights on the axles of one of the heaviest electric locomotives that has been built. Bridges over which such heavy locomotives are to pass, or over which it is likely they will pass in the future, should be designed for Cooper's E50, in the same way as other railroad bridges, or for the actual loads, as explained above.

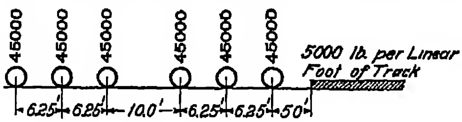


Fig 12

237. On electric roads designed for the multiple-unit system, which uses no locomotives, each car carrying its own motors, it is necessary to ascertain if there is any possibility of the road being used in the future by steam or electric locomotives; if so, due allowance should be made so that the bridges will be strong enough for them. If it is likely that nothing but the cars will ever run on the road, the bridges should be designed for the actual weights of the cars when fully loaded, increased 10 or 20 per cent to provide for a possible increase in the weight of future cars. Fig 13 rep-

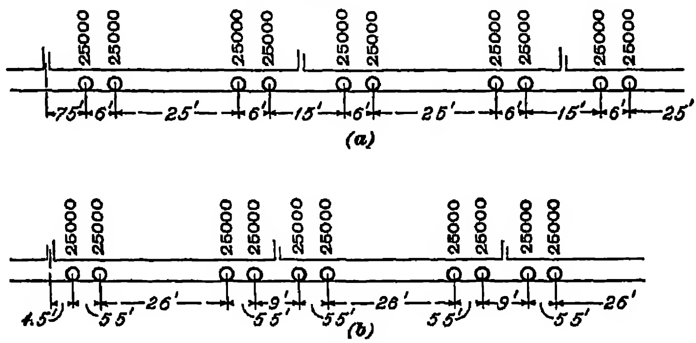


FIG. 13

resents the weights and axle spacing of the cars run by the multiple-unit system on two different roads. The specifications given for railroad bridges should be used for such system of loading, except that the new loading should be substituted for that given in Art. **24.**

238. The loads to which highway bridges are subjected differ considerably from those to which railroad bridges are subjected. They usually consist of heavy snowfalls, crowds of people, wagons, road rollers, and street cars. It has been observed that city bridges are more likely to be subjected to the weight of large crowds of people and of wagons than country bridges; a crowd of people may cover the entire floor of a short span, but is not likely to cover the whole floor of a long span. For this reason, the uniform load per square foot that is used is heavier for city than for country bridges, and heavier for short than for long spans.

Heavier road rollers are used on city streets than on country roads. The case of street cars, however, is somewhat different. The heaviest cars run in city streets are, as a rule, those that are used for interurban traffic; that is, to connect cities. These cars cross country bridges as well as city bridges, and both kinds of bridges must be designed to support them. The loads given in Art. 98 represent, as near as possible, those to which the different bridges may be subjected, and are approximately the same as those now in use by the best engineers. It is not probable that they will increase for a great many years.

239. Impact and Vibration.—As explained in *Strength of Materials*, Part 1, a load that is suddenly applied produces twice as great a stress as one that is gradually applied, and a load that is applied in a very short interval of time causes a stress greater than that caused by a load that is gradually applied, but less than that caused by a suddenly applied load. The moving loads that cross bridges are applied in a short interval of time, more so in the case of railroad than of highway bridges, and it is customary to make some allowance for the resulting increased stresses in the members and for the shock and vibration of the bridge under the passing loads. The increase in the stresses is called the **effect of impact and vibration.** Some experiments have been made and several formulas proposed for determining the magnitude of this increase. Owing, however, to the difficulty and expense of making experiments, it has been impossible to obtain reliable results. Practice varies considerably in respect to the formula used; some engineers use two values for the allowable intensity of stress - one for dead-load stress, and the other, usually one-half the former, for live-load stress. This method has the disadvantage that it causes difficulty in designing, especially in compression members; it is necessary to find separately the areas of cross-section required to resist the dead-load and the live-load stresses and to add them in each case to obtain the required dimensions. This method is, besides, illogical, as it tacitly

assumes that the proportional effect of impact and vibration is the same in all members, while as a matter of fact it is greater in such members as floor members and hip verticals, which receive their maximum live-load stresses in a short interval of time (in some cases, $\frac{1}{4}$ second), than in such members as chords of long spans, which do not receive their maximum live-load stresses in so short a time. The best engineers allow the same intensity of stress for both dead-load and live-load stresses, but add a certain amount to the live-load stress as calculated from the loading. In some cases, the amount added is a percentage of the live load equal to the ratio of the live- to the sum of the live- and dead-load stresses; but in the great majority of cases allowance is made as specified in Arts. **25** and **99**. The time required for any live-load stress to rise from zero to its maximum depends on the time it takes the moving load to cover the part of the bridge that must be loaded in order to cause the maximum stress. The formulas just referred to take this into account; they are the results of experience, and are the most satisfactory so far devised. More allowance is made in railroad bridges than in highway bridges that carry electric railways, as in the former class of bridges the loads move much faster and also cause more shock and vibration.

EXAMPLE 1.—The live-load stress in the center panel of the upper chord of a railroad bridge truss 150 feet long is 280,000 pounds. What amount must be added for impact and vibration?

SOLUTION.—In this case, S = 280,000, as the member under consideration is a chord member, the entire span must be loaded to produce the stress S. Then (see Art. **25**), L = 150, and, therefore,

$$I = \frac{300}{150 + 300} \times 280{,}000 = 186{,}700 \text{ lb.} \quad \text{Ans.}$$

EXAMPLE 2.—In a highway bridge truss 125 feet long, the live-load stress in the end post, due to the load on the car track, is 75,000 pounds. What amount must be added for impact and vibration?

SOLUTION.—In this case, S = 75,000, as the member under consideration is an end post, the entire length of span must be loaded to produce the stress S. Then (see Art. **99**), L = 125, and, therefore,

$$I = \frac{300 - 125}{1{,}000} \times 75{,}000 = 13{,}100 \text{ lb.} \quad \text{Ans.}$$

240. Reversal of Stress.—It is a well-known fact, established by experiment, that a piece of metal that is subjected to a large number of repetitions of varying stresses will finally break, even though the greatest stress to which it has been subjected is much less than the ultimate strength of the metal. This is true whether the stresses are of the same kind or are of opposite kinds Bridge members are subjected to a great number of repetitions of varying stresses, and various methods are in use to make allowance for the effect. Some of these methods make use of different allowable intensities of stresses in different members, the actual value in any case depending on the ratio of the minimum to the maximum stress. It is the best practice at the present time, however, to ignore the effect in those members in which the maximum and minimum stresses are of the same kind, as the range of stress is comparatively small, and to allow for it in those members in which the maximum and minimum stresses are of opposite kinds, as the range of stress is then comparatively large.

The customary way to make allowance in those members in which the stress reverses is to add to each stress eight-tenths of the other and then design the member for both of the increased stresses. For example, if the maximum stress in a member is 25,000 pounds compression, and the minimum stress is 10,000 pounds tension, the member must be designed for

$$25{,}000 + \tfrac{8}{10} \times 10{,}000 = 33{,}000 \text{ pounds compression}$$

and for

$$10{,}000 + \tfrac{8}{10} \times 25{,}000 = 30{,}000 \text{ pounds tension}$$

EXAMPLE —The maximum stress in a member is 50,000 pounds tension, and the minimum stress 20,000 pounds compression. (*a*) If the allowable intensity of tensile stress is 16,000 pounds per square inch, what is the required net section of the member? (*b*) If the value of $\frac{l}{r}$ is such that the allowable intensity of compressive stress is 13,500 pounds per square inch, what is the required gross section?

SOLUTION.—(*a*) The tension for which the member is to be designed is $50{,}000 + \tfrac{8}{10} \times 20{,}000 = 66{,}000$ lb.

Then, the required area of net section is

$$66,000 \div 16,000 = 4.125 \text{ sq. in.} \quad \text{Ans.}$$

(*b*) The compression for which the member is to be designed is

$$20,000 + \tfrac{8}{10} \times 50,000 = 60,000 \text{ lb.}$$

Then, the required gross area is

$$60,000 \div 13,500 = 4.444 \text{ sq. in.} \quad \text{Ans.}$$

241. Dead Load.—In finding the dead load, it is customary first to decide on the type of floor and calculate its weight. Then the approximate weight of the steelwork can be calculated by some formula. The weight of a bridge per linear foot depends on the moving load for which the bridge is designed and on the intensities of stress adopted. Formulas for calculating weights of bridges are purely empirical; they are based on values that have been observed in actual structures. On account of the fact that railroad bridges are, as a rule, designed for one loading, the formulas for their weights are fairly reliable. In the case of highway bridges, in which the loadings and widths vary within a wide range, it is almost impossible to give formulas that represent all the conditions. In any event, it is necessary, after having completed the design of a bridge, to compute its weight accurately; if the computed weight differs much from the assumed weight, the dead-load stresses should be recomputed and the cross-sections of the members altered to correspond with the corrected stresses. This is sometimes spoken of as **revising the design.**

242. Weights of Railroad Bridges.—The following formulas give the weights w, in pounds per linear foot, of the steel work in railroad bridges designed according to the specifications from Arts. 14 to 90 for Cooper's E50, l being the span, in feet:

Rolled I beams, $w = 25\,l$ for each track.

Deck plate-girder bridges, $w = 500 + 8\,l$ for one track.

Half-through plate-girder bridges, $w = 800 + 12\,l$ for one track.

Riveted truss bridges,

$$w = 1,500\left[1 + \left(\frac{l-90}{100}\right)^2\right] \text{ for one track}$$

Riveted truss bridges,

$$w = 2{,}600\left[1 + \left(\frac{l - 90}{100}\right)^2\right] \text{ for two tracks}$$

Pin-connected truss bridges may be assumed 5 per cent. lighter than riveted truss bridges for spans over 200 feet in length. For spans shorter than 200 feet, the same formulas may be used as for riveted truss bridges.

The above weights are for bridges having standard-tie floors, as described in Art. **48.** Solid-floor bridges will, in general, be about 25 per cent. heavier.

243. Weights of Highway Bridges.—The following formulas give the weights w, in pounds per linear foot, of one girder or truss in highway bridges designed according to the specifications given in Arts **91** to **159;** l being the span, in feet, and W the maximum load per linear foot supported by the girder or truss, including the live load together with the impact, and the weight of floorbeams, stringers, railings, and floor.

Plate-girder bridges, $w = \frac{W}{1{,}000}(24 + .8\,l)$.

Riveted truss bridges, $w = \frac{W}{12}\left[1 + 2\left(\frac{l - 90}{100}\right)^2\right]$.

Pin-connected truss bridges may be assumed 5 per cent. lighter than riveted truss bridges for spans over 200 feet in length. For spans shorter than 200 feet, the same formulas may be used as for riveted truss bridges.

DESIGN OF PLATE GIRDERS

(PART 1*)

GENERAL PRINCIPLES

BEAMS

1. Introduction.—The principles governing the distribution of stress in the cross-section of a beam, and the formulas by which the stresses are obtained from the moments and shears, have been explained in connection with the theory of beams in *Strength of Materials*. The formula for the maximum tensile and compressive stresses at any section of a beam is

$$s = \frac{Mc}{I}$$

in which s = maximum intensity of stress;

c = distance of most remote part of section from neutral axis;

I = moment of inertia of entire cross-section about neutral axis;

M = bending moment at section considered.

If the expression $\frac{I}{c}$, which is the section modulus, is represented by Q, the formula for the maximum intensity of stress becomes

$$s = \frac{M}{Q}$$

*All the tables referred to in this Section are given in *Bridge Tables* and explained in *Bridge Members and Details*, Parts 1 and 2. In referring to the Section entitled *Bridge Specifications*, the title will for convenience be abbreviated to *B S*.

From this formula follows

$$Q = \frac{M}{s}$$

Care must be taken that the proper units are used in these formulas. It is customary to express s in pounds per square inch, c in inches, and M in inch-pounds; the moment of inertia I and section modulus Q found from the dimensions of the section expressed in inches.

2. I Beams.—The maximum intensity of stress in an I beam is found by the formula $s = \frac{M}{Q}$. The section moduli of I beams of different weights and depths are given in Table XIV. The practical problem, however, usually consists in finding what size of I beam will safely carry a given load.

To solve this problem, the maximum bending moment M is computed, and then, by means of the formula $Q = \frac{M}{s}$, the required value of the section modulus is found. An I beam having a section modulus equal to or slightly greater than that required is then chosen from Table XIV. The same method is followed in designing channels to be used as beams.

EXAMPLE 1.—The bending moment at a given section of a 12-inch 40-pound I beam is 672,000 inch-pounds What is the maximum intensity of stress at that section?

SOLUTION —Consulting Table XIV, the section modulus of a 12-in. 40-lb I beam is found to be 44 8. Then, since M is 672,000 in.-lb., the maximum intensity of stress s is

672,000 ÷ 44.8 = 15,000 lb per sq. in Ans

EXAMPLE 2 —The maximum bending moment on a beam is 1,840,000 inch-pounds If it is required that the maximum intensity of stress shall not exceed 16,000 pounds per square inch, what size and weight of I beam must be used?

SOLUTION —Since M is 1,840,000 in -lb., and s is 16,000 lb per sq. in , the required value of section modulus Q is 1,840,000 ÷ 16,000 = 115 Consulting Table XIV, and following from the smaller beams toward the larger, the first I beam that has a section modulus as large as required is found to be a 15-in. 95-lb. beam, which has a section

modulus of 116 4. Looking lower down, however, it is found that a 20-in 65-lb beam has a section modulus of 117, which is also sufficient. The latter beam should be used, as it is 30 lb per ft lighter, and therefore more economical than the 95-lb beam.

NOTE —The advantage of using a deep beam is here apparent, as the required value of section modulus can be had with a lighter beam than if a shallower beam were used.

EXAMPLES FOR PRACTICE

1 The bending moment at a given section of a 24-inch 100-pound I beam is 3,472,000 inch-pounds. What is the maximum intensity of stress at that section? Ans. 17,500 lb per sq. in.

2 The bending moment at a given section of an 18-inch 65-pound I beam is 1,370,600 inch-pounds. What is the maximum intensity of stress at that section? Ans 14,000 lb per sq. in.

3. The maximum bending moment on an I beam is 133,300 foot-pounds What size I beam must be used in order that the maximum intensity of stress shall not exceed 16,000 pounds per square inch? Ans A 20-in 65-lb I beam

4 The maximum bending moment on an I beam is 400,000 inch-pounds What size I beam must be used in order that the maximum intensity of stress shall not exceed 16,000 pounds per square inch? Ans. A 10-in. 30-lb. I beam

PLATE GIRDERS

SECTION MODULUS

3. The maximum stresses due to bending moment in a plate girder may be found by means of the formula $s = \frac{M}{Q}$. As the flanges are not the same size throughout the whole length of the girder, however, the value of Q changes wherever the section of flange changes, and in order to use the formula it is necessary to compute the value of Q at every section where the flange changes. This requires much time and is rarely done in practice; a modification of the foregoing formula is used that gives sufficiently close results and is much more convenient. This formula will now be explained.

4. Let Fig. 1 be a vertical section of a plate girder having dimensions as follows: thickness of web, t; width (depth) of web, h, vertical distance between centers of gravity of flanges, h_g; total height or depth of section, h_1, gross area of cross-section of top flange and net area of cross-section of bottom flange, which will be assumed equal, A. It will be assumed that the cross-section of the girder is symmetrical; then, the neutral axis will be at the center, at a distance $\frac{h_g}{2}$ from the center of gravity of each flange. As $Q = \frac{I}{c}$, it is necessary to compute the value of I; c is equal to $\frac{h_1}{2}$.

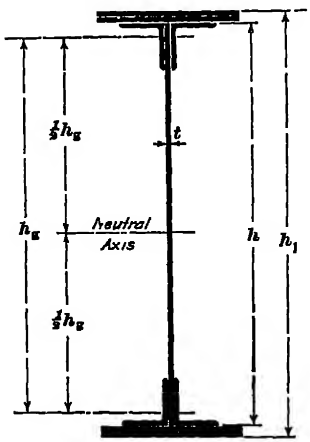


FIG 1

The moment of inertia of the entire cross-section about the neutral axis is the sum of the moment of inertia of the web about the neutral axis, the moment of inertia of each flange about an axis parallel to the neutral axis and passing through the center of gravity of the flange, and the product of the area of each flange and the square of the distance from its center of gravity to the neutral axis.

The moment of inertia of the web about the neutral axis is $\frac{t h^3}{12}$. As part of the web is cut out by rivet holes, it is customary to allow for the decrease in strength by using for the moment of inertia three-fourths of the theoretical value; that is, in this case, $\frac{3}{4} \times \frac{t h^3}{12} = \frac{t h^3}{16}$. The moment of inertia of each flange about an axis through its center of gravity is very small compared with the other terms, and in practice it is customary to neglect it. The product of the area of each flange and the square of the distance of the center of gravity of the flange from the neutral axis is $A \times \frac{h_g^2}{4}$. For

both flanges, twice this value, or $\frac{A h_g^2}{2}$, should be taken. Therefore, for the moment of inertia of the entire cross-section of the girder about the neutral axis, we have, approximately,

$$I = \frac{A h_g^2}{2} + \frac{t h^3}{16}$$

whence, since $Q = \frac{I}{c}$, and $c = \frac{h_1}{2}$,

$$Q = \frac{\frac{A h_g^2}{2} + \frac{t h^3}{16}}{\frac{h_1}{2}} = \frac{A h_g^2}{h_1} + \frac{t h^3}{8 h_1}$$

This formula is not convenient in this form. By assuming that in the first term h_1 can be replaced by h_g, and that in the second term h_1 can be replaced by h, and h^3 by $h^2 \times h_g$, the following more convenient, and sufficiently approximate formula is found:

$$Q = \frac{A h_g^2}{h_g} + \frac{t h^2 h_g}{8 h} = A h_g + \frac{t h h_g}{8} = h_g\left(A + \frac{t h}{8}\right)$$

As a matter of fact, h, h_1, and h_g are as a rule very nearly equal, and there is not very much error in assuming that any of these quantities can be replaced by either of the other two.

EXAMPLE.—The web of a plate girder is 48 in. $\times \frac{1}{2}$ in. in cross-section. Each flange has an area of section equal to 20 square inches, and the vertical distance between their centers of gravity is 47 inches. What is the section modulus of the cross-section?

SOLUTION.—In this case, $h_g = 47$ in., $A = 20$ sq. in., $t = \frac{1}{2}$ in., and $h = 48$ in. Substituting these values in the formula,

$$Q = 47 \times \left(20 + \frac{48 \times .5}{8}\right) = 1{,}081 \quad \text{Ans.}$$

DESIGN OF FLANGES

5. Determination of Flange Area.—Since $s = \frac{M}{Q}$, we have also, $M = s Q$. Substituting for Q its value given in Art. **4**, this equation becomes

$$M = s h_g\left(A + \frac{t h}{8}\right);$$

whence $$\frac{M}{s\,h_g} = A + \frac{t\,h}{8}$$

and $$A = \frac{M}{s\,h_g} - \frac{t\,h}{8}$$

The expression $\frac{M}{s\,h_g}$ is sometimes spoken of as the flange area, and the term $\frac{t\,h}{8}$ is spoken of as the portion of web that goes to make up the flange area, or that assists in resisting the bending moment.

In designing, the distance between the centers of gravity of the flanges is usually first assumed equal to the width of web, and the flange is designed on that basis, then, the correct distance is calculated, and, if necessary, the areas of the flanges are changed to correspond with it.

6. Effect of Web.—It is sometimes specified that the flanges shall be considered as resisting the entire bending moment, without considering the assistance of the web. In this case, as the last expression in the formula for A in Art. **5** represents the effect of the web, it is simply necessary to omit it. The resulting formula is

$$A = \frac{M}{s\,h_g}$$

By using this formula, the area of each flange is made larger than necessary by an amount equal to one-eighth the cross-section of the web. This assumption evidently gives incorrect results; but, as it provides more flange area than is required, it is on the safe side The only objection to it is that it is not economical. In the following articles, it will be assumed that the web assists in resisting bending moment.

EXAMPLE —A plate girder having a 60″ × $\frac{3}{8}$″ web is subjected to a maximum bending moment of 2,000,000 foot-pounds If the allowable intensity of bending stress is 16,000 pounds per square inch, what is the trial value that would be used for the area of each flange: (*a*) if the web is assumed to assist in resisting the bending moment? (*b*) if the flanges are assumed to resist the entire bending moment?

SOLUTION.—(*a*) As the bending moment is given in foot-pounds, it must be multiplied by 12, in order to reduce it to inch-pounds. This gives $M = 12 \times 2{,}000{,}000 = 24{,}000{,}000$ in.-lb. The value of s

is 16,000 lb per sq in As the trial values of the areas are required, the trial value of h_g, that is, the width of the web, or 60 in, will be used In the first case, the required area of the flange is given by the formula in Art **5**,

$$A = \frac{M}{s\,h_g} - \frac{t\,h}{8}$$

Here, $t = \frac{3}{8}$ in., and $h = h_g = 60$ in. Then, substituting in the formula,

$$A = \frac{24{,}000{,}000}{16{,}000 \times 60} - \frac{.375 \times 60}{8} = 25 - 2.81 = 22.19 \text{ sq in.} \quad \text{Ans}$$

(*b*) In the second case, the required area of flange is given by the formula $A = \frac{M}{s\,h_g}$ Substituting the given values,

$$A = \frac{24{,}000{,}000}{16{,}000 \times 60} = 25 \text{ sq. in} \quad \text{Ans.}$$

7. Length of Flange Members.—As the bending moment near the end of a girder is less than at the center, the required area of flange section is less near the end than at the center For this reason, each flange is composed of several parts, usually two angles and one or more plates The angles, and in some cases one plate, are continued the entire length of the girder; the other plates are shorter, and are cut off where they are no longer required. It is very difficult to find, by the analytic method, the sections at which the different flange plates are no longer required; a combination of the analytic and the graphic method is more convenient. The required areas of flange at several sections along the girder are computed by means of the formula $A = \frac{M}{s\,h_g}$, and a **curve of flange areas** is drawn. Fig. 2 shows the curves for the top and bottom flanges of a plate girder, and the graphic method of determining the sections at which the plates are no longer required. As plate girders are usually symmetrical about the center, only one-half of the span is shown. The diagram is explained in the following article.

8. Curve of Flange Areas.—On any line $X'X$, Fig. 2, the distance $R\,O$ is laid off to scale equal to one-half the span, and the sections A, B, C, etc., at which the flange areas have been computed, are marked in their proper positions. At A, B, C, etc., lines are drawn at right angles to $X'X$, and on

them are laid off to scale the distances AA', BB', CC', etc., above and below $X'X$ to represent the values of A as found by the formula $A = \frac{M}{s\,h_g}$ If the load on the girder is concentrated at the points A, B, C, etc., as in a half-through plate-girder bridge, straight lines RA', $A'B'$, $B'C'$, etc. are drawn connecting the points just found; if the load on the girder is

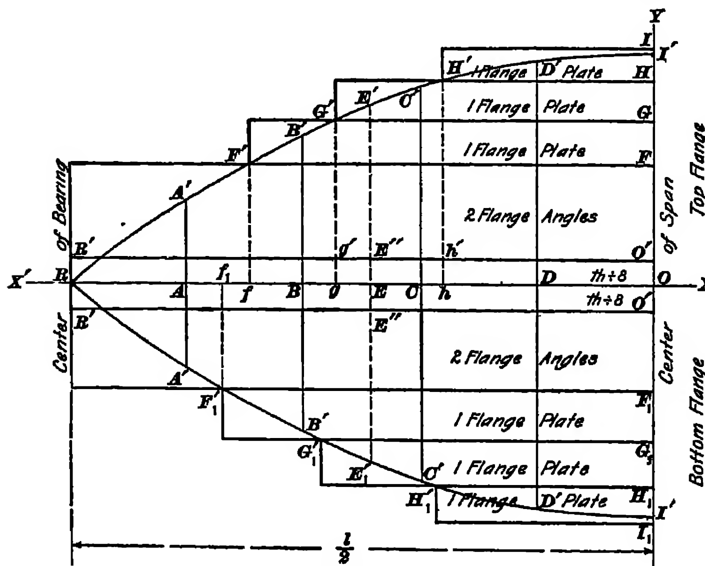


FIG 2

distributed, as in a deck plate-girder bridge, smooth curves are passed through the points R, A', B', C', D', and I', above and below the line $X'X$, respectively. In the present case, it has been assumed that the load is uniformly distributed; then, the curves RI' are the curves of flange areas, and the ordinate at any section, such as EE' at E, represents the required flange area at that section. The next step in the construction of the diagram consists in laying off to scale on the line YOY' at right angles to $X'X$ at O the areas of the different sections that make up the flanges. When it is specified that the effect of the web in assisting to resist the bending moment is to be neglected, the areas of

the angles and plates are laid off directly from O, when it is specified that the effect of the web is to be considered, as in the present case, the points O' are located on $Y\,O\,Y'$ at such a distance from O that $O\,O'$ represents to scale $\frac{t\,h}{8}$, and the lines $O'\,R'$ are drawn parallel to $O\,R$ Then, the ordinates between the lines $O'\,R'$ and the curves, at any section, such as $E''\,E'$ at E, represent the required area that must be provided in the angles and plates at that section. In the present case, it will be assumed that two angles and three plates are required at the center in each flange; $O'\,F$ represents the gross area of the two angles, and $F\,G$, $G\,H$, and $H\,I$, the gross areas of the three plates in the top flange; $O'\,F_1$, $F_1\,G_1$, $G_1\,H_1$, and $H_1\,I_1$ represent the net areas of the angles and plates in the bottom flange. Lines are drawn through the points F, G, H, etc. to their intersections F', G', H', etc , respectively, with the curve; then, f, g, etc., on perpendiculars from F', G', etc to $X'\,X$, are the sections at which the different flange plates are no longer required.

For example, it is seen that at the section h the required area of the top flange is represented by $h'\,H'$, which is equal to $O'\,H$, and this represents the sum of the areas of the two angles and two plates. Hence, as the outside plate is not required, it can be discontinued at H', and $H\,H'$ will represent one-half its length. In a similar manner, at the section g the required area of the top flange is represented by $g'\,G'$, which is equal to $O'\,G$, and this represents the sum of the areas of the two angles and one plate. Hence, as the two outside plates are not required, the second plate can be discontinued at G' By the same method, the section at which any plate is no longer required can be found when there are any number of plates.

EXAMPLES FOR PRACTICE

1 The web of a plate girder is 84 in. $\times \frac{9}{16}$ in. in cross-section Each flange has an area of section equal to 36 square inches, and the vertical distance between their centers of gravity is 83 inches. What is the section modulus of the cross-section? Ans. 3,478

2 A plate girder having a 56″ × ½″ web is subjected to a maximum bending moment of 1,792,000 foot-pounds If the allowable intensity of bending stress is 16,000 pounds per square inch, what is the trial value that would be used for the area of cross-section in the design of each flange: (*a*) if the web is assumed to assist in resisting the bending moment? (*b*) if the flanges are assumed to resist the entire bending moment?

Ans $\begin{cases} (a) & 20\ 5 \text{ sq in} \\ (b) & 24 \text{ sq. in} \end{cases}$

3 The required flange areas at several sections of a deck plate girder 64 feet long are as follows: at 8 feet from the end, 21 square inches, at 16 feet from the end, 36 square inches, at 24 feet from the end, 45 square inches, and at the center of the span, 48 square inches. The upper flange is made up as follows

	Square Inches
$th \div 8$	= 4 12
Two angles, 6 in × 6 in × ¾ in @ 8.44 . . .	= 16 88
Three plates, 16 in × $\frac{7}{16}$ in @ 7 00	= 21.00
One plate, 16 in. × ⅜ in @ 6 00 . . .	= 6 00
	48 00

What are the theoretical lengths of the four flange plates to the next larger whole foot? Ans. 23 ft , 34 ft , 42 ft., 48 ft

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DESIGN OF WEB

9. Longitudinal Shearing Stress.—The distribution of stress in the cross-section of a beam can be represented as in Fig. 3, in which the lines with arrowheads represent the intensities of stress at different distances from the neutral axis. All the stresses acting on the section above the neutral axis are in one direction; all below, in the other direction. On this account, there is a tendency for the portion of the beam on one side of the neutral plane to slide horizontally on the portion on the other side. The same is true at any horizontal section, such as $x\,x$, the part above $x\,x$ tending to slide on or shear away from the part below the plane $x\,x$. This tendency causes a shearing stress called the **longitudinal shearing stress** or **longitudinal shear,**

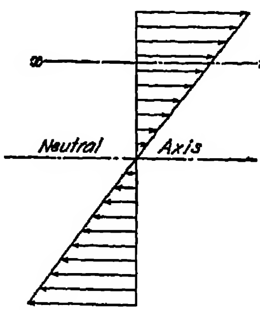


Fig 3

which decreases as the distance from the neutral axis ncreases, is greatest at the neutral axis, and is zero at the outside of the section It can be shown by advanced mathematics that the longitudinal shearing stress in a beam is given by the formula

$$s_1' = \frac{VG}{I} \qquad (1)$$

n which s_1' = longitudinal stress, in pounds *per linear inch* of beam (that is, the stress that would occur in a length of the beam equal to 1 inch, if the stress in all that portion had the same intensity as at the section considered);

V = maximum vertical shear, in pounds, on the section considered;

I = moment of inertia, about the neutral axis, of the entire section, derived from the dimensions of the cross-section in inches;

G = static moment, about neutral axis, of all that part of section outside of point considered (that is, the product of the area of that part of the section and the distance from its center of gravity to the neutral axis).

If the thickness of the beam is denoted by t, the intensity s_1 of longitudinal shear, per square inch, is equal to $s_1' \div t$; that is,

$$s_1 = \frac{s_1'}{t} = \frac{VG}{tI} \qquad (2)$$

In calculating longitudinal shear, it is immaterial whether he vertical shear is positive or negative, as only the numercal value is required.

10. Variation in Shearing Stress.—As the vertical shear V is greater near the end than at the center, the longiudinal shear is also greater at the end than at the center. Considering a vertical section of the beam, V and I are constant for that section; that is, they do not vary, no matter what part of the section is considered; G decreases as the point at which the longitudinal shear is desired is taken

farther from the neutral axis, and is zero at the outside of the section. Therefore, according to formula **2**, Art. **9**, *the maximum intensity of longitudinal shear occurs at the neutral axis, near the end of the span.*

11. Web Shear.—It can be shown that the intensity of vertical shearing stress at any point in the cross-section of a beam is equal to the intensity of longitudinal shear at the same point. The maximum intensity of vertical shear at any section of a plate girder can therefore be found by applying the formula $s_1 = \frac{VG}{tI}$ to a point at the neutral axis of the section. In Art. **4**, it was found that the value of I is approximately $\frac{A h_g^2}{2} + \frac{t h^3}{16}$. The static moment G is the sum of the static moment of the area A of one flange and the static moment of the area $\frac{t h}{2}$ The distance of the center of gravity of A from the neutral axis is $\frac{h_g}{2}$; and, therefore, the static moment of A is $A \times \frac{h_g}{2}$ The distance of the center of gravity of the area of one-half the web from the neutral axis is $\frac{h}{4}$, and, therefore, the static moment of that area is

$$\frac{t h}{2} \times \frac{h}{4} = \frac{t h^2}{8}$$

Therefore, $$G = \frac{A h_g}{2} + \frac{t h^2}{8}$$

Substituting in formula **2**, Art **9**, the values just found for I and G, there results

$$s_1 = \frac{V\left(\frac{A h_g}{2} + \frac{t h^2}{8}\right)}{t\left(\frac{A h_g^2}{2} + \frac{t h^3}{16}\right)} \qquad (1)$$

As the terms $\frac{t h^2}{8}$ and $\frac{t h^3}{16}$ are small, compared to the other terms, they may be omitted without any error worth considering in practice. Formula **1** then becomes

$$s_1 = \frac{V \times \frac{A\,h_g}{2}}{t \times \frac{A\,h_g^2}{2}} = \frac{V}{t\,h_g}$$

or, approximately, since h_g is very nearly equal to h,

$$s_1 = \frac{V}{t\,h} \qquad (2)$$

The product $t\,h$ is equal to the area of cross-section of the web. Formula **2** may, therefore, be stated in the form of a rule as follows:

Rule.—*To find the maximum intensity of shearing stress in the web of a plate girder at any section, divide the maximum vertical shear, in pounds, by the gross area of cross-section of the web in square inches.*

12. Stiffeners.—In *Bridge Members and Details*, it was explained that a flat plate, such as the web of a plate girder, tends to buckle when subjected to a shearing stress, and for this reason it is stiffened by angles riveted to the sides of the plate. The holes for the rivets in the stiffeners decrease the cross-section of the plate, and it has been found in practice that the effective or net section through such a row of holes is very nearly 75 per cent. of the gross section. To allow for the decrease in section, it is customary to specify that the intensity of shearing stress found by the formula $s_1 = \frac{V}{t\,h}$ at any point shall not exceed 75 per cent. of the allowable intensity of shearing stress.

Example 1.—The maximum vertical shear at a given section of a plate girder is 180,000 pounds, and the area of cross-section of the web is 30 square inches What is the maximum intensity of shearing stress?

Solution.—In this case, V is 180,000 lb. and $t\,h$ is 30 sq. in. Substituting in formula **2**, Art **11**,

$$s_1 = 180{,}000 \div 30 = 6{,}000 \text{ lb. per sq. in.} \quad \text{Ans}$$

Example 2.—The maximum vertical shear at a given section of a girder is 240,000 pounds. If the width of web is 72 inches, and the clear unsupported distance at the given section is 26 inches, what thickness must be used in order that the intensity of shearing stress shall not exceed the values given in Table XXXVI?

Solution —In an example of this kind, which is common, it is necessary to assume either the intensity of shearing stress or the thickness of web The experienced designer can, as a rule, assume the thickness of web pretty close to the actual thickness, and later correct his assumption In the present case, the intensity cannot exceed 9,000 lb per sq in (see Table XXXVI), therefore, the cross-section of the web cannot be less than 240,000 ÷ 9,000 = 26 667 sq in., and the thickness cannot be less than this divided by the height, or 26 667 ÷ 72 = .37 in., say $\frac{3}{8}$ in. The area of cross-section of a 72″ × $\frac{3}{8}$″ plate is 27 sq. in.; then, the actual intensity of shear, if a $\frac{3}{8}$-in plate is used, will be 240,000 ÷ 27 = 8,888 lb per sq in Consulting Table XXXVI, it is found that the allowable intensity of shear in a $\frac{3}{8}$-in plate with an unsupported width of 26 in is 4,600 lb per sq in As this is so much less than the actual intensity, a thicker plate must be tried The difference between the actual and the allowable intensity being so great, the next thickness, $\frac{7}{16}$ in , will not be considered, but a plate $\frac{1}{2}$-in. thick will be tried

The area of cross-section of a 72″ × $\frac{1}{2}$″ plate is 36 sq in , then, the actual intensity of shear, if a $\frac{1}{2}$-in. plate is used, will be 240,000 – 36 = 6,667 lb. per sq. in. Consulting Table XXXVI, it is found that the allowable intensity of shear is 6,400 lb. per sq. in As this is less than the actual intensity, a thicker plate must be used The next thickness, that is, $\frac{9}{16}$ in . will be tried. The area of cross-section of a 72″ × $\frac{9}{16}$″ plate is 40 5 sq in , then, the actual intensity of shear, if a $\frac{9}{16}$-in plate is used, will be 240,000 ÷ 40 5 = 5,930 lb per sq in Consulting Table XXXVI, it is found that the allowable intensity of shear is 7,000 lb per sq in. As this is greater than the actual intensity, the assumed thickness is sufficient. Therefore, a 72″ × $\frac{9}{16}$″ plate may be used.

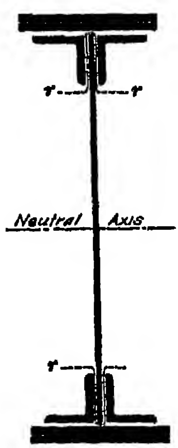


Fig. 4

13. Pitch of Flange Rivets.—The longitudinal shear in a beam of solid cross-section is resisted by the shearing strength of the material. That in a beam built up of various simple rolled sections is resisted by the rivets that hold the different parts together so as to make them act as one piece. The longitudinal shearing stress in a plate girder tends to cause the flange angles to slide on the web. In finding this stress, it is customary to consider a section, such as *r r*, Fig. 4, made between the flange angles and the web, this section cuts

only the rivets that connect the vertical legs of the flange angles to the web. The longitudinal shearing stress on the section rr, per unit of length, is found by the formula in Art. **9**,

$$s_1' = \frac{VG}{I}$$

In this case, G is simply the static moment of the flange about the neutral axis, or $\frac{A h_g}{2}$. Substituting this value for G, and using the value of I found in Art **4**, the formula becomes

$$s_1' = \frac{V\left(\frac{A h_g}{2}\right)}{\frac{A h_g^2}{2} + \frac{t h^3}{16}}$$

or, neglecting in the denominator the term $\frac{t h^3}{16}$, which is very small compared with $\frac{A h_g^2}{2}$,

$$s_1' = \frac{V\left(\frac{A h_g}{2}\right)}{\frac{A h_g^2}{2}} = \frac{V}{h_g}$$

If the pitch of the flange rivets is p, then, since the longitudinal shear per unit of length is s_1', the stress K that is transmitted by one rivet is $p s_1'$, or $p \times \frac{V}{h_g}$. In computing rivet pitch, K is the value of one rivet. We have, therefore,

$$K = \frac{pV}{h_g} \qquad (1)$$

and

$$p = \frac{K h_g}{V} \qquad (2)$$

As h_g does not materially differ from the distance h_r between the rivet lines in the vertical legs of the flange angles, it is the general practice, and one that will be followed in this Course, to replace h_g by h_r in the preceding formulas. Those formulas then become

$$K = \frac{pV}{h_r} \qquad (3)$$

$$p = \frac{K h_r}{V} \qquad (4)$$

The distance h_r is always less than h_g, and so formula **4** gives a smaller pitch than is actually required; that is, the error is on the side of safety.

EXAMPLE 1.—At a given section of a plate girder, the maximum vertical shear is 72,000 pounds, the distance between the rivet lines of the flanges is 27 inches, and the pitch of the flange rivets is 3 inches How much stress is transmitted by each rivet?

SOLUTION.—Substituting the given values in formula **3**,

$$K = \frac{3 \times 72{,}000}{27} = 8{,}000 \text{ lb} \quad \text{Ans.}$$

EXAMPLE 2 —The maximum vertical shear at a given section of a plate girder is 150,000 pounds, the value of a flange rivet is 9,600 pounds, and the vertical distance between the rivet lines of the flanges is 37.5 inches. What is the required pitch of the rivets?

SOLUTION —To use formula **4**, we have $V = 150{,}000$ lb , $h_r = 37.5$ in., and $K = 9{,}600$ lb Substituting these values in the formula, we have

$$p = \frac{9{,}600 \times 37\ 5}{150{,}000} = 2\ 4 \text{ in.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. The maximum vertical shear at a given section of a girder is 240,000 pounds The width of web is 64 inches, and the thickness is $\frac{9}{16}$ inch What is the maximum intensity of shearing stress?
Ans 6,667 lb per sq in.

2. The maximum vertical shear at a given section of a girder is 100,000 pounds. If the width of web is 36 inches, and the clear unsupported distance is 24 inches, what thickness must be used in order that the intensity of shearing stress shall not exceed $\dfrac{12{,}000}{1 + \dfrac{d^2}{3{,}000\,t^2}}$?
Ans $\frac{1}{2}$ in.

3. The maximum vertical shear at a given section of a plate girder is 102,000 pounds; the distance between the rivet lines of the flanges is 34 inches; and the pitch of the flange rivets is 2 5 inches How much stress is transmitted by each rivet? Ans 7,500 lb

4 The maximum vertical shear at a given section of a plate girder is 178,200 pounds; the value of a flange rivet is 10,800 pounds; and the vertical distance between the rivet lines of the flanges is 49 5 inches. What is the required pitch of rivets? Ans. 3 in.

GENERAL DESIGN

14. Deck and Half-Through Girder Bridges.—The design of a plate girder requires the calculation of the maximum shears and moments at several sections along the girder.

In a half-through plate-girder bridge, the load is applied to the girders at the panel points, and it is simply necessary to calculate the maximum bending moment at each panel point, and the maximum shear in each panel. The bending moment may be assumed to vary uniformly between panel points, that is, the part of the moment curve between two panel points may be assumed to be straight. The shear in each panel may be assumed constant between panel points; then, the intensity of the shearing stress and the rivet pitch will be constant in each panel, but will change at each panel point.

In a deck plate-girder bridge, the load is applied at every point throughout the length of the girder, and it is necessary to compute the maximum moments and shears at several sections along the girder; for convenience, these sections are usually taken from 5 to 10 feet apart. From the values so found, the intensity of the shearing stress, the rivet pitch, and the required flange areas at the different sections are found by means of the formulas already given. The values at intermediate sections can then be found by means of a curve similar to the curve of flange areas.

15. Girders With Curved and Inclined Flanges. The same formulas are used for girders with curved or inclined flanges as for those with parallel flanges. As, however, the height of the girder is different at different sections, the width of the web h, and the vertical distance h_g between the centers of gravity of the flanges, must be calculated at each section at which the flange area, intensity of shearing stress, or rivet pitch is required. The results will be true for the horizontal flange, and for all practical purposes near enough to the true results for the inclined flange, unless the angle between the inclined flange and the horizontal is greater

than about 10°. In the latter case, the required area of section of the inclined flange at any section may be found by multiplying the area as found by the formula $A = \frac{M}{s h_g} - \frac{t h}{8}$ (Art. 5) by the secant of the angle between the inclined flange and the horizontal. The other values are close enough for all practical purposes

16. Girders With Vertical Flange Plates and Secondary Flange Angles.—In long girders in which the flanges must be very large, it is advisable to make use of a modified type of flange in order to avoid having a great thickness of flange plates, and consequently an excessive

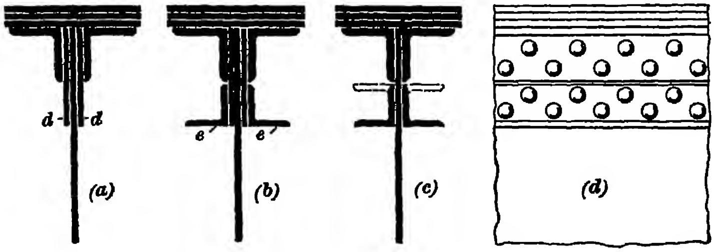


FIG 5

grip for the rivets. The modified flange usually has one of the forms shown in Fig 5 (*a*), (*b*), and (*c*). In Fig. 5 (*a*), vertical flange plates *d*, *d*, each of which has a width about twice that of the flange angle, are inserted between the flange angles and the web. In Fig. 5 (*b*), there are, in addition to the vertical flange plates, secondary flange angles *e*, *e* riveted to the flange plates below the main flange angles. In Fig. 5 (*c*), the secondary flange angles are used and are riveted to the web, the vertical flange plates being omitted. The secondary flange angles are sometimes placed with the outstanding leg below, as shown in full lines, and sometimes with it above, as shown by dotted lines in Fig 5 (*c*). The design of a girder with flanges as represented in Fig 5 makes use of no new principle; the formulas and methods that have been explained in the preceding articles are used. In calculating the location of the center of gravity of the flange

the vertical plates and secondary angles must be taken into consideration.

17. The vertical plates, as well as the main flange angles, are continued the entire length of the girder; the secondary flange angles are usually stopped where they are no longer required. To find the required length of flange members, the curve of flange areas is used as represented in Fig. 6.

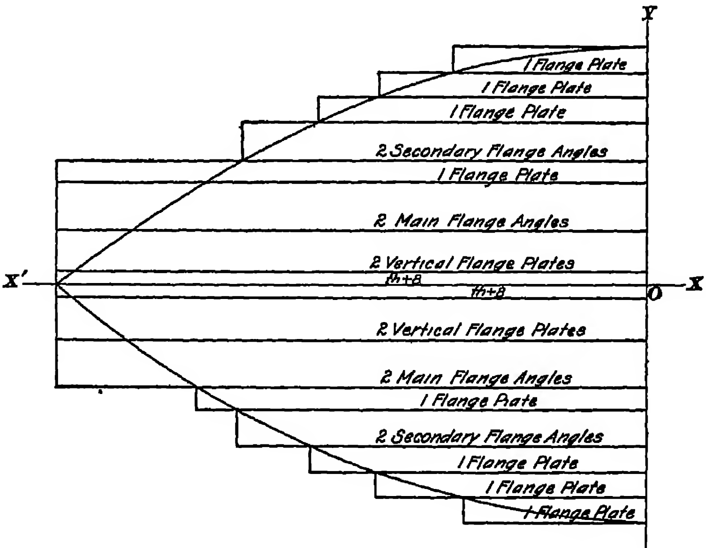


FIG 6

The value $\frac{t\,h}{8}$ is first laid off on $Y\,O\,Y'$ above and below $X\,O\,X'$; then the area of the two vertical flange plates, then that of the two main flange angles, then the first flange plate; then the secondary flange angles; and then the remaining flange plates, in order.

SPLICES

18. Lengths of Members.—The web and the flange members require to be spliced when they are longer than the lengths given in Table V. Web-plates are generally made as long as possible. Flange plates are made narrower than

web-plates, and can be procured in greater lengths; angles also can be procured in great lengths, in many cases as long as 100 feet. Owing to practical difficulties in handling long slender pieces, however, it is usually specified that all flange members shall be spliced when they are longer than about 70 feet.

19. Web Splices.—The different portions of which the web is composed are made the full height of the girder and rectangular, with the ends at right angles to the flanges. The ends of two consecutive portions are placed together and covered by plates, called **splice plates**, on each side. These plates are riveted to the ends of the portions of the web in the manner represented in Fig 7 (*a*), in which *A B* is the section at which the web is spliced, and the plate *C D* is one of the splice plates. The flange angles assist somewhat in splicing the web, but it is customary to ignore their effect, and to design the splice on the assumption that the web is spliced entirely by the splice plates, and to make the splice as strong as the web.

If the splice is designed to resist the same bending moment as the web, there will be no necessity to calculate the shear on the joint, as the joint will always be strong enough in shear. The bending moment M that the web can bear, sometimes called the **resisting moment**, is given by the formula

$$M = \frac{sI}{c} = \frac{s \times \frac{t h^2}{16}}{\frac{h}{2}} = \frac{s t h^2}{8},$$

using for the moment of inertia the approximate value $\frac{t h^2}{16}$ (see Art. 4). If h_1 represents the height and t_1 the combined thickness of the two splice plates, one on each side of the web, $\frac{s t_1 h_1^2}{8}$ represents the combined resisting moment of the two splice plates. In order that they may have the same resisting moment as the web, we must have

$$\frac{s t_1 h_1^2}{8} = \frac{s t h^2}{8};$$

whence

$$t_1 = t \frac{h^2}{h_1^2}$$

By means of this formula, the required thickness of the splice plates can be found. It is seldom necessary, however, to compute the required thickness; it is generally specified that each plate shall have a sectional area at least 75 per cent. that of the web, in which case there is invariably sufficient section.

20. Resisting Moment of Web-Splice Rivets.—The stress in the web is transmitted to the splice plates by vertical rows of rivets. In Fig. 7 (*a*), two vertical rows transmit the stress to the splice plates on one side of the splice,

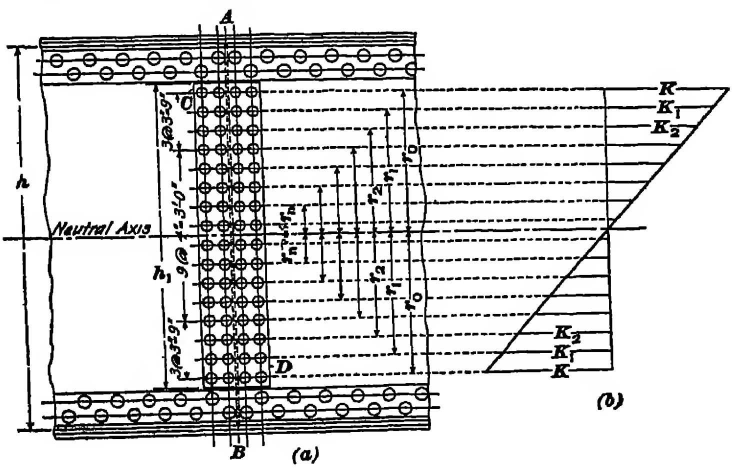


FIG 7

and two rows transmit the stress from the splice plates on the other side The rivets on each side of the splice must have the same resisting moment as the web. The intensity of stress due to bending varies uniformly as the distance from the neutral axis; in like manner, the amount of stress transmitted by a rivet varies as its distance from the neutral axis, being greatest for the rivet whose distance from the neutral axis is greatest, as represented in Fig. 7 (*b*). If K represents the stress on one outside rivet, at a distance r_0 from the neutral axis, then $K_1 = K \times \frac{r_1}{r_0}$ represents the stress on any other rivet at a distance r_1 from the neutral

axis The resisting moment of a rivet is the product of the stress on the rivet and its distance from the neutral axis, as $K r_0$, $K_1 r_1$, $K_2 r_2$, etc. Expressing the stress on each rivet in terms of K, the resisting moments of the rivets are

$$K r_0 = \frac{K r_0^2}{r_0},\ K_1 r_1 = K \frac{r_1}{r_0} r_1 = \frac{K r_1^2}{r_0},\ K_2 r_2 = \frac{K r_2^2}{r_0}, \text{ etc.}$$

Therefore, for the combined resisting moment M_r of all the rivets on one side of the splice, we have

$$M_r = \frac{K r_0^2}{r_0} + \frac{K r_1^2}{r_0} + \frac{K r_2^2}{r_0} + \ldots = \frac{K}{r_0}(r_0^2 + r_1^2 + r_2^2 + \ldots)$$

or, denoting by Σr^2 the sum $r_0^2 + r_1^2 + r_2^2 + \ldots$ of the squares of the distances r_0, r_1, r_2, etc.,

$$M_r = \frac{K}{r_0} \Sigma r^2 \qquad (1)$$

It should be clearly understood that, in applying this formula, the distance of *every rivet* from the neutral axis should be squared, and the results added. When several rivets are at the same distance from the neutral axis, as is the case when rivets are arranged in horizontal rows, or when some rivets are as far above as others are below the neutral axis, the work is much simplified by squaring that distance and multiplying the result by the number of rivets to which the distance is common. Thus, in Fig. 7 (*a*), each distance, as r_1, is common to four rivets, two above and two below the neutral axis. In this case,

$$\Sigma r^2 = 4(r_0^2 + r_1^2 + r_2^2 + \ldots)$$

In order that the splice rivets may have sufficient strength, the value of M_r must be at least as great as the resisting moment of the web, and K must not exceed the value of the outside rivet. We must, therefore, have

$$\frac{K}{r_0} \times \Sigma r^2 = \frac{s t h^2}{8} \qquad (2)$$

In applying this formula, it is customary to assume first a spacing of rivets and calculate their resisting moment. If the value is not sufficient, more rivets are added, either by spacing the rivets closer together, or by adding another row outside of the first two. For this purpose, the arrangement represented in Fig. 8 is frequently employed, the

advantage being that most of the rivets are near the flanges, where the value of the resisting moment is greatest. The thickness of these plates may be found by means of the formula $t_1 = t \times \frac{h^2}{h_1^2}$ (Art. **19**), substituting for h_1 the sum of the heights of the three portions of the splice plate; the thickness on each side, however, is usually made about 75 per cent. of that of the web.

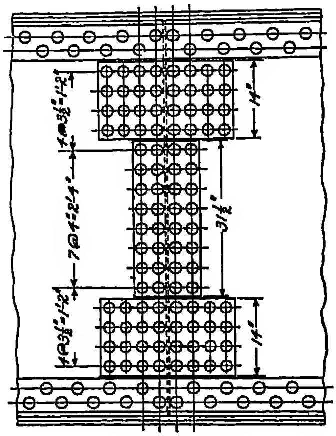


FIG. 8

EXAMPLE.—If the plate girder represented in Fig. 8 has a web 72 in. × ½ in., spliced with ⅞-inch rivets and splice plates as shown, what is (*a*) the required thickness of splice plates on each side? (*b*) the resisting moment of the rivets on each side of the splice, assuming the value of a rivet to be 9,600 pounds?

SOLUTION.—(*a*) As the splice plates are made up of three parts on each side of the web, the value of h_1 is $14 + 31\frac{1}{2} + 14 = 59.5$ in. Then, since t is .5 and h is 72 in.,

$$t_1 = .5 \times \frac{72^2}{59.5^2} = .5 \times 1.46 = .73 \text{ in.}$$

for both sides, or 73 − 2 = 365 in. (say ⅜ in.) for each side. Ans.

(*b*) The distances of the rivets from the neutral axis are 2, 6, 10, 14, 17½, 21, 24½, and 28 in., respectively. There are four rivets at each of the first four distances, and eight rivets at each of the last four. To apply formula **1**, we have $K = 9{,}600$ lb., $r = 28$ in., and

$$\Sigma r^2 = 4 \times (2^2 + 6^2 + 10^2 + 14^2) + 8 \times (17.5^2 + 21^2 + 24.5^2 + 28^2) = 18{,}396$$

Therefore,

$$M_r = \frac{9600}{28} \times 18{,}396 = 6{,}307{,}200 \text{ in.-lb. Ans.}$$

21. Flange-Angle Splices.—Flange angles are spliced by riveting angles to them, as represented in Fig. 9. In this figure, (*a*) is the elevation of a portion of the bottom flange, (*b*) is the cross-section at *B B*, and (*c*) is the plan of the flange and a cross-section at *C C*. The angles *d* are the flange angles, the angles *e* are the splice angles, and the line *f* is the section at which the flange angle is spliced. Practice

varies as to the method of designing a flange-angle splice. Some engineers use one splice angle riveted to the flange angle that is spliced; this is open to the objection that the splice angle must be very long in order to get sufficient rivets to develop the stress, and therefore interferes with other details, such as stiffeners. Others use two angles, as in Fig. 9, and assume that the stress in the flange angle is equally divided between the two. On account of the fact that the angle on the side opposite the splice gets its stress through the web and the other flange angle, it is probable that the angle in contact with the flange angle that is spliced

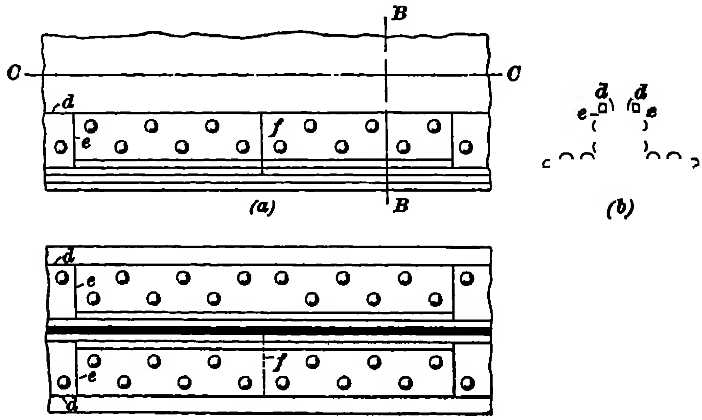


FIG 9

gets somewhat more than one-half the stress. The assumption most frequently made is that the splice angle in contact with the flange angle that is spliced takes three-quarters of the stress; this angle is designed on this basis to have a cross-section three-quarters that of the flange angle, and made long enough to get sufficient rivets on each side of the splice to transmit three-quarters of the stress. The other splice angle is then made the same size and length. If F_A is the area of section (net area for tension, gross area for compression) of one flange angle, then the area F_A' of section of each splice angle is given by the formula

$$F_A' = .75\, F_A \qquad (1)$$

If s is the allowable intensity of bending stress, the total stress in the flange angle is $F_A s$, and in the splice angle, $.75 F_A s$. The number n of rivets in the splice angle on each side of the splice is given by the formula

$$n = \frac{.75 F_A s}{K} \qquad (2)$$

in which K is the value of one rivet. Splice angles are cut from angles having legs the same width as the flange angles, so that the edges of splice and flange angles will be even, and not as shown by dotted lines in Fig. 9 (b). The area of cross-section of such an angle can be found by deducting from the area given in Tables IX and X for the original angle the amount that is sheared off.

EXAMPLE.—The flange angles in the tension flange of a plate girder are 6 in. × 6 in. × $\frac{1}{2}$ in., and the allowable intensity of stress is 16,000 pounds per square inch. The diameter of the rivets is $\frac{7}{8}$ inch. (a) What size of splice angle must be used if the angle is spliced as represented in Fig. 9? (b) If the value of one rivet is 5,000 pounds, how many rivets are required on each side of the splice?

SOLUTION.—(a) The area of a $6'' \times 6'' \times \frac{1}{2}''$ angle is 5.75 sq. in. As we are considering the tension flange, the net section is required, as there are two rivet holes in each section, and the area of a hole for a $\frac{7}{8}$-inch rivet is .5 sq. in., as given in Table XXVII, the net section is $5.75 - 2 \times .5 = 4.75$ sq. in. ($= F_A$). The required net area F_A' of one splice angle is, then, by formula 1, $.75 \times 4.75 = 3.56$ sq. in. As the flange angle is $\frac{1}{2}$ in. thick, $\frac{1}{2}$ in. must be sheared off each leg of each splice angle. The thinnest $6'' \times 6''$ angle, which is $\frac{3}{8}$ in. thick, will be tried first. The area of this angle, as given in Table IX, is 4.36 sq. in.; if $\frac{1}{2}$ in. is cut from each leg, the area is reduced by $2 \times .5 \times .375 = .37$ sq. in., nearly. As there are two holes in each angle, the area will be still further reduced by $2 \times .375 = .75$ sq. in. Then, the net area of one angle $5\frac{1}{2}$ in. × $5\frac{1}{2}$ in. × $\frac{3}{8}$ in. will be $4.36 - .37 - .75 = 3.24$ sq. in. As 3.56 sq. in. is required, this is not enough. The next size, 6 in. × 6 in. × $\frac{7}{16}$ in., the gross area of which is 5.06 sq. in., will be tried. If $\frac{1}{2}$ in. is sheared from each leg, the area is reduced by $2 \times .5 \times .44 = .44$ sq. in. The two rivet holes still further reduce the section by $2 \times .44 = .88$ sq. in. Then, the net area of one angle $5\frac{1}{2}$ in. × $5\frac{1}{2}$ in. × $\frac{7}{16}$ in. is $5.06 - .88 - .44 = 3.74$ sq. in., which is sufficient. Ans.

(b) The total stress in one flange angle is $4.75 \times 16{,}000 = 76{,}000$ lb., and the portion to be transmitted by the rivets is three-fourths of this,

or 57 000 lb Then, the required number of rivets is 57,000 ÷ 5,000 = 11 4, or, say, 12 rivets on each side of the splice.

22. Flange-Plate Splices.—Flange plates are spliced either by additional plates, or by continuing the outer plates beyond their theoretical ends a sufficient distance to splice the plates below them. The plates nearest the flange angles are the longest, and are the ones that require to be spliced.

23. Additional Splice Plates.—In the first method of splicing, the joint in the plate that is to be spliced is usually

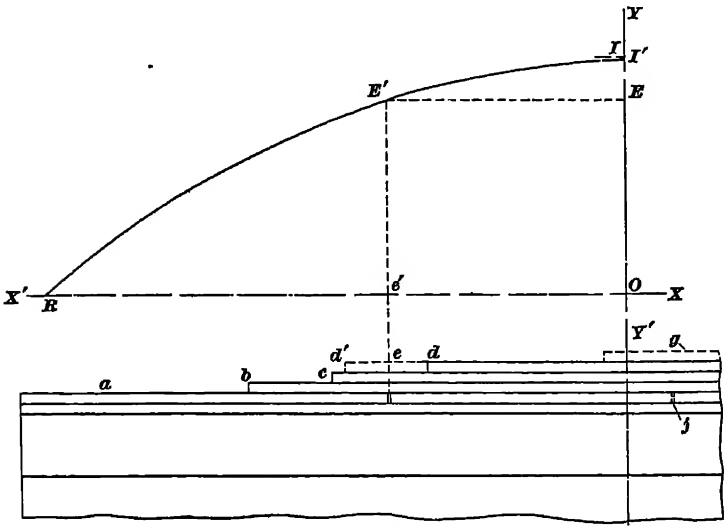


Fig 10

located somewhere near the center of the girder, as represented at *j*, Fig. 10, and a splice plate *g* of the same thickness as the flange plate is riveted to the outside of the flange. The splice plate is made long enough to contain sufficient rivets on each side of the joint to properly transmit the stress from one part of the flange plate to the other. The number of rivets that is required to transmit the stress to or from the splice plate can be found by dividing the stress by the value of one rivet, the latter will usually be the value in single shear. When, as in Fig. 10, there are intermediate

plates between the splice plate and the plate that is spliced, there is some uncertainty as to how the stress is carried around the joint; to provide for this, the number of rivets in the splice plate is increased as specified in Arts. **61** and **134** of *B. S.* In the present case, since there are three intermediate plates, the number of rivets will be increased $3 \times 20 = 60$ per cent. Any flange plate may be spliced in the same way.

24. Continuing Outer Plates.—In the second method of splicing, the location of the joint in the plate that is to be spliced is found by means of the curve of flange areas. In Fig 10, the curve of flange areas and one-half of the flange are laid out to the same horizontal scale, the points d, c, and b are the theoretical ends of the three outside flange plates, and it is desired to splice the first flange plate. The ordinate $O I'$ represents the flange area as found by the formula $A = \frac{M}{s\,h_g}$ (Art. **6**), and $O I$ represents the actual area of flange. From I, $I E$ is laid off to scale to represent the area of the plate that is to be spliced, the line $E E'$ is drawn parallel to $X' O X$ to its intersection E' with the curve, and the line $E' e$ is drawn perpendicular to $X' O X$. Then, $e' E'$ represents the entire area of flange, exclusive of the plate a; that is, if all the plates are carried beyond e, the first plate a can be spliced at that section. For this purpose, the outer plate, instead of being stopped at d, is carried beyond e, as represented by the dotted lines. As there is no stress in the first flange plate a at the section e, and there is full stress in the outer plate d, it remains to find how many rivets must be contained in the plate d beyond e, in order to transmit its stress to a; this is found by dividing the stress in the outside plate by the value of one rivet in single shear. In the present case, as there are two plates between d and a, the required number of rivets will be $2 \times 20 = 40$ per cent. greater than the theoretical number. The plate d will be continued to a point d', the distance $e d'$ being made such that there will be sufficient room for the required number

of rivets. In a similar manner, any flange plate may be spliced.

It will seldom be found necessary to splice any flange plate at more than one section, nor more than two plates in any flange. When two plates must be spliced, the first plate can be spliced at one end of the girder, as at e, Fig. 10, and the second plate at a corresponding point on the other side of the center.

EXAMPLE.—Let Fig 10 represent the top flange of a plate girder in which the allowable intensity of stress is 16,000 pounds per square inch and the sizes of the plates are as follows a, 16 in. $\times \frac{1}{2}$ in.; b, 16 in $\times \frac{1}{2}$ in , c, 16 in $\times \frac{7}{16}$ in ; and d, 16 in $\times \frac{3}{8}$ in (*a*) If it is desired to splice plate a at section j, what is the required size of the splice plate g? (*b*) If the value of one rivet in single shear is 6,600 pounds, how many rivets must be included in plate g? (*c*) If it is desired to splice plate a at section e, how many rivets must be included in plate d between e and d', assuming the value of one rivet to be 6,600 pounds?

SOLUTION —(*a*) As plate g is an additional splice plate, it must be the same size as the plate that is to be spliced, that is, 16 in $\times \frac{1}{2}$ in. Ans

(*b*) The area of cross-section of plate g is 8 sq in , and, since the intensity of stress is 16,000 lb per sq in , the stress in plate g is $8 \times 16{,}000 = 128{,}000$ lb As the value of one rivet in single shear is 6,600 lb , the number of rivets required to transmit this stress to plate g is $128{,}000 \div 6{,}600 = 19.4$ Since there are three intermediate plates between g and a, the number must be increased $3 \times 20 = 60$ per cent. Then, the total number of rivets required on each side of j is

$$19.4 + \tfrac{60}{100} \times 19.4 = 31$$

As the flange rivets in the plates are driven in pairs, it is necessary to have 32 rivets Ans

(*c*) The area of cross-section of plate d is 16 in $\times \frac{3}{8}$ in. = 6 sq. in.; and, since the intensity of stress is 16,000 lb. per sq. in., the stress in plate d is $6 \times 16{,}000 = 96{,}000$ lb. As the value of one rivet is 6,600 lb., the number of rivets required to transmit the stress to the plate d is $96{,}000 \div 6{,}600 = 14.5$ rivets Since there are two intermediate plates between d and a, the number must be increased $2 \times 20 = 40$ per cent. Then, the total number of rivets required in plate d between d' and e is

$$14.5 + \tfrac{40}{100} \times 14.5 = 20.3$$

As this is so close to 20, 20 rivets will be sufficient, although it is better to use 22 Ans

25. Splices in Secondary Flange Angles and Vertical Flange Plates.—Secondary flange angles are usually

spliced, as represented in Fig. 11, by means of one splice angle riveted to the inside of the flange angle and a splice plate having a width about the same as that of the flange angle, riveted to the back of the latter. The area of the splice angle is made about 75 per cent. and that of the splice plate about 25 per cent. that of the flange angle, the area of the two together being not less than the area of the flange angle.

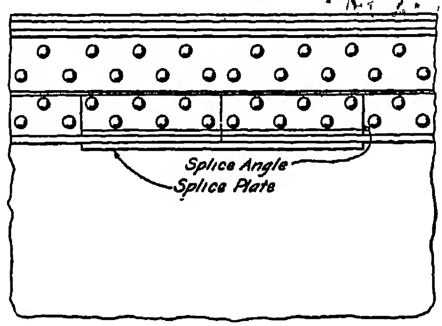


FIG. 11

Vertical flange plates are spliced, as represented in Fig. 12, by means of one vertical splice plate riveted to the vertical legs of the flange angles on the same side of the web as the vertical plate that is spliced. The area of the splice plate is made not less than that of the flange plate.

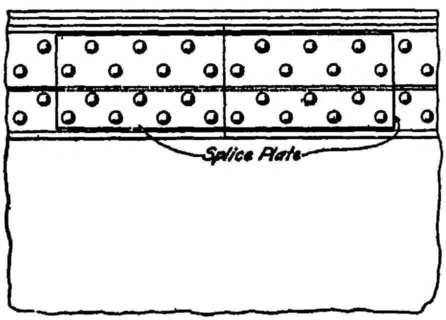


FIG. 12

EXAMPLES FOR PRACTICE

1. What is the resisting moment of a 36″× $\frac{5}{8}$″ web, if the maximum intensity of the bending stress is 16,000 pounds per square inch?

Ans. 1,620,000 in.-lb.

2. If the web represented in Fig. 7 is 70 inches wide and $\frac{9}{16}$ inch thick, what is the required thickness of splice plate on each side, assuming the height to be 57.5 inches? Ans. $\frac{7}{16}$ in. thick on each side

3. If the rivets in Fig. 7 are spaced as shown at the left-hand side, and the value of one rivet is 10,800 pounds, what is the resisting moment of the rivets in the splice? Ans. 3,849,600 in.-lb.

4. The flange angles in the compression flange of a plate girder are 8 in $\times$ 8 in $\times \frac{3}{4}$ in, and the allowable intensity of stress is 16,000 pounds per square inch (*a*) What size of splice angle must be used? (*b*) If the value of one rivet is 6,600 pounds, how many rivets are required in the splice angle?

Ans $\begin{cases} (a) \text{ 2 angles, 8 in} \times \text{8 in} \times \frac{5}{8}\text{ in, cut to } 7\frac{1}{4}\text{ in} \times 7\frac{1}{4}\text{ in} \times \frac{5}{8}\text{ in} \\ (b) \text{ 22 rivets on each side of the joint} \end{cases}$

5 (*a*) If the vertical flange plate in Fig 12 is 15 inches wide and $\frac{1}{2}$ inch thick, and the splice plate is 13 inches wide, what is the required thickness of the latter? (*b*) If the allowable intensity of stress is 16,000 pounds per square inch, and the value of one rivet is 6,600 pounds, how many rivets are required in the splice plate?

Ans. $\begin{cases} (a) \ \frac{5}{8}\text{ in thick} \\ (b) \text{ 18 or 20 rivets on each side of the joint} \end{cases}$

BEARINGS

26. Size of Bedplates.—The end of a girder, where the girder rests on the masonry or other support, must be

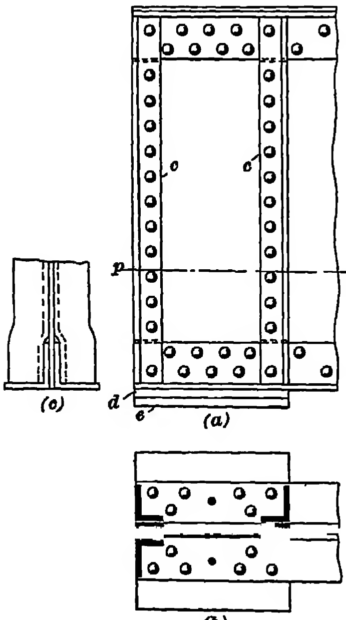


FIG 13

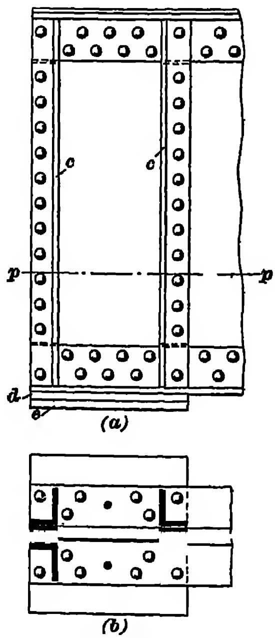


FIG 14

strong enough to resist the reactions. The required area of

bearing, if the girder rests on masonry, is found by dividing the maximum reaction by the allowable intensity of bearing on the masonry. For example, if the reaction is 200,000 pounds, and the allowable intensity of bearing is 500 pounds per square inch, the required area of bearing is 200,000 ÷ 500 = 400 square inches. Bedplates are usually made rectangular and very nearly square; in the present case, each will need to be about $\sqrt{400}$ = 20 inches on each side.

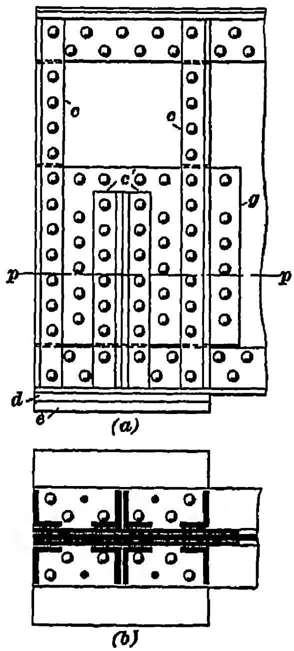


FIG. 15

27. End Stiffeners.—The ends of plate girders are stiffened by stiffeners at each end of the bedplates. The arrangements usually employed are represented in Figs 13, 14, and 15, in which *c*, *c* are the stiffeners, *d*, the sole plates, and *e*, the bedplates The arrangement represented in Fig. 13 is most used, although that shown in Fig. 14 is somewhat better on account of the fact that in it the bearing of the stiffeners is not so close to the edge of the bedplate; the pressure is therefore more evenly distributed over the area of the bedplate. In the arrangement represented in Fig. 15, the additional stiffeners *c'* are added The plates *g* are called **reinforcing plates**, and are added to distribute the stress more evenly over the web.

28. Distribution of Reaction.—The reaction is assumed to be equally divided among the end stiffeners, and evenly distributed over the area of the outstanding legs of the stiffeners where they bear on the upper side of the lower flange angle. The length of the portion of a stiffener in contact with the lower flange angle is usually about ½ inch less than the nominal width of the outstanding leg. If *t'* is

the thickness of a stiffener angle, b, the nominal width of the outstanding leg, and s_b, the allowable intensity of bearing on the end of a stiffener, the stress that one stiffener can resist is $t'(b - \frac{1}{2})s_b$. If R is the reaction and n the number of end stiffeners, the amount of pressure that is transmitted by each stiffener is $\frac{R}{n}$, and, in order that the stiffener may be sufficiently strong, the following equation must be satisfied:

$$t'(b - \tfrac{1}{2})s_b = \frac{R}{n}$$

From this equation, we have

$$t' = \frac{R}{n\, s_b(b - \frac{1}{2})},$$

from which the required thickness of the stiffeners can be found. The nominal width of leg is controlled by Arts. 55 and 128 of *B. S.*

29. Rivets and Stiffeners.—The stress in each stiffener is transmitted to the web by means of rivets. The required number of rivets in a stiffener can be found by dividing the stress in one stiffener $\left(\frac{R}{n}\right)$ by the value of a rivet in single shear, or in bearing on the angle, or, since the same rivets connect two opposite angles, by dividing the stress in two stiffeners $\left(\frac{2R}{n}\right)$ by the value of a rivet in double shear, in bearing on two angles, or in bearing on the web, whichever is least. In practice, it is considered advisable to transmit the greater part of the stress in the stiffeners to the web below the neutral axis. When it is impossible to get sufficient rivets below the neutral axis in four stiffeners, as in Figs. 13 and 14, more stiffeners are used, as in Fig. 15.

30. Crimped Stiffeners.—Stiffeners are sometimes placed in contact with the web, and the ends **crimped**; that is, bent out around the vertical legs of the flange angles, as represented in Fig. 13 (*c*). This arrangement is open to the objection that it is difficult to bend the angles so they will bear evenly on the flange angle, especially on the outstanding leg.

31. Loose Fillers.—The best practice at the present time is to make the stiffener angles straight from top to bottom, and fill in the space between them and the web by means of bars the same width as the adjacent leg of the stiffener and the same thickness as the flange angles, as represented in Figs. 13 and 14. These bars are called **loose fillers**; they simply serve to fill the space between the angle and the web. The number of rivets required to connect the stiffener to the web must be increased 20 per cent. when this form of filler is used.

32. Reinforcing Plates or Tight Fillers.—The filler under the stiffeners in Fig. 15 is continued under all the angles, and riveted to the web by rivets located outside of the stiffeners. The filler is then called a **tight filler**, or **reinforcing plate**; it distributes the stress over the area of the web. Such a plate is not to be considered an intermediate plate, but rather a part of the web, as it is firmly riveted to it, and in calculating the bearing value of the rivet on the web the thickness of these plates must be included.

EXAMPLE 1.—The maximum reaction at the end of the plate girder represented in Fig 13 is 135,000 pounds (*a*) If the stiffeners are 5 in. × $3\frac{1}{2}$ in., and the allowable intensity of bearing is 18,000 pounds per square inch, what is the required thickness of the stiffener angles? (*b*) If the value of one rivet in single shear is 6,600 pounds, in bearing on the web, 10,800 pounds, and in bearing on each stiffener angle, 8,400 pounds, how many rivets are required to connect the stiffeners to the web?

SOLUTION.—(*a*) The required thickness of stiffeners can be found by the formula in Art. 28,

$$t' = \frac{R}{n\,s_b\,(b - \frac{1}{2})}$$

In the present case, $R = 135{,}000$, $n = 4$, $s_b = 18{,}000$, and $b = 5$. Substituting these values in the formula gives

$$t' = \frac{135{,}000}{4 \times 18{,}000 \times 4.5} = .42 \text{ in., or } \tfrac{7}{16} \text{ in., nearly. Ans.}$$

(*b*) The rivets that connect the stiffeners to the web are in single shear on each side of the web, and in bearing on the $\frac{7}{16}$-inch stiffener angle. Considering first the stress in one angle, the value in single shear is evidently less than the value in bearing on the $\frac{7}{16}$-inch angle.

The stress in each stiffener is $135,000 \div 4 = 33,750$ lb., and the required number of rivets is $33,750 \div 6,600 = 5.1$

Considering now the stress in two angles, the rivets are in double shear at $2 \times 6,600 = 13,200$ lb.; in bearing on two $\frac{7}{16}$-inch angles, at $2 \times 8,400 = 16,800$ lb.; and in bearing on the web, at 10,800 lb. The latter value being the smallest, and the stress in two stiffeners being $135,000 \div 2 = 67,500$ lb., the required number of rivets is $67,500 \div 10,800 = 6.25$. This number is larger than that first found, and must be used. As there are loose fillers (see Art. **31**), the actual number required is

$$6.25 + \frac{20}{100} \times 6.25 = 7.5, \text{ say, } 8 \text{ rivets} \quad \text{Ans.}$$

EXAMPLE 2.—The maximum reaction at the end of the plate girder represented in Fig. 15 is 230,000 pounds, the thickness of web is $\frac{1}{2}$ inch, and the thickness of the flange angles is $\frac{5}{8}$ inch. Using the working stresses given in Art. **29** of *B. S.*, it is desired to find: (*a*) the required thickness of stiffeners, if the outstanding leg is 5 inches wide, (*b*) the number of rivets required to connect the stiffeners to the web, if $\frac{7}{8}$-inch rivets are used.

SOLUTION.—(*a*) In Art. **29** of *B. S.*, the allowable intensity of bearing on the ends of stiffeners is given as 18,000 lb. per sq. in. Then, since $R = 230,000$, $n = 8$, and $b = 5$, we have, applying the formula in Art. **28**,

$$t' = \frac{230,000}{8 \times 18,000 \times 4.5} = .355, \text{ or, say, } \tfrac{3}{8} \text{ in.} \quad \text{Ans.}$$

(*b*) Considering a single stiffener, the value in bearing on a $\frac{3}{8}$-inch angle, as given in Table XL, is 7,220 lb., and in single shear, 6,600 lb. The latter is the smaller. The stress in one stiffener is $230,000 \div 8 = 28,750$ lb., and the required number of rivets is $28,750 \div 6,600 = 4.4$.

Taking two stiffeners, it is necessary to consider the value of the rivet in double shear at 13,230 lb., in bearing on two stiffeners at 14,440 lb., and in bearing on the combined thickness of web and two reinforcing plates $1\frac{3}{4}$ in. The last is evidently greater than either of the other two values, the value in double shear is the smallest, and must be used. Since the stress in two stiffeners is twice that in one, and the value in double shear is twice that in single shear, the number of rivets will be the same as in the first case considered, that is, 4.4, or, say, 5 rivets in each stiffener. Ans.

33. Rocker Bearings.—The ends of spans over 75 feet in length are supported on rockers and supplied at one end with rollers, as explained in *Bridge Members and Details*. When it is necessary, on account of high water, to keep the bridge seat close to the bottom of the girders, the

arrangement represented in Fig 16 is used. The outstanding legs of the lower flange angles are cut away for a short distance at each end to allow the lower flange to enter between the vertical plates of a pedestal The web of the

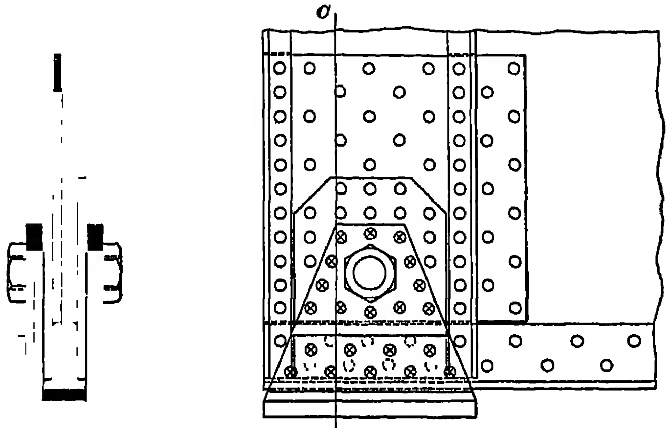


FIG 16

girder is reinforced at the end, and a pin is passed through the pedestal and web; the pin should never be less than 6 inches in diameter. The design of the pin, pin plates, and pedestal is made by the same general principles that apply to pin-connected trusses, as explained in another Section.

EXAMPLES FOR PRACTICE

1 If the maximum reaction at the end of the plate girder represented in Fig. 13 is 157,000 pounds, stiffeners 5 in. × $3\frac{1}{2}$ in , and allowable intensity of bearing 18,000 pounds per square inch, what is the required thickness of the stiffener angles? Ans $\frac{1}{2}$ in

2. If in example 1 the value of a rivet in single shear is 6,600 pounds, and in bearing on the web 15,000 pounds, how many rivets must be used in each stiffener to transmit the stress to the web?

Ans 8 rivets

3 If the maximum reaction at the end of the plate girder represented in Fig. 15 is 275,000 pounds, stiffeners 5 in. × $3\frac{1}{2}$ in , and allowable intensity of bearing 18,000 pounds per square inch, what is the required thickness of the stiffener angles? Ans. $\frac{7}{16}$ in

DESIGN OF PLATE GIRDERS

(PART 2)

DESIGN OF AN I-BEAM HIGHWAY BRIDGE

1. Introduction.—In this and in the following Sections will be given complete designs of several classes and types of bridges. The designs will be made according to the rules given in *Bridge Specifications* (a title that, for convenience, will be abbreviated to *B. S.*). These examples will familiarize the student with the principles involved and the methods used, which he can without any difficulty extend to forms and conditions not specifically covered in the present instruction.

2. Data.—As a first example, an I-beam highway bridge will be designed from the data given in the **data sheet** on page 4. The words in *Italics* are supposed to have been written to fill out the general form, which contains only the words printed in Roman type (see *B. S.*, Art. **226**).

3. Determination of Span.—It is first necessary to determine the location of the abutments. According to *B. S.*, Art. **18**, no part of a bridge should be less than 7 feet from the center line of the nearest track, nor less than 22 feet above the base of the rail. This condition applies also to abutments and underneath clearance lines for overhead bridges. As the faces of abutments are usually rough and extend somewhat beyond the neat lines, it is well to locate the neat lines at the base of the rail 7 feet 6 inches from the

center line of the track, making them (as there are two tracks 13 feet center to center) 7 feet 6 inches + 13 feet + 7 feet 6 inches = 28 feet apart. The faces of abutments are sometimes made plumb (vertical), in order to shorten

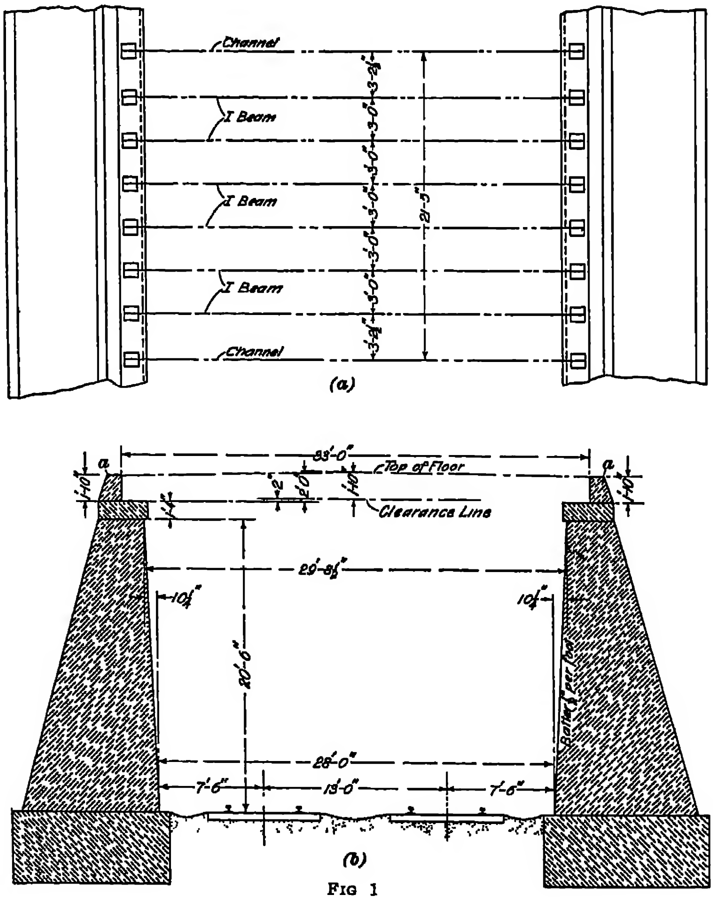


Fig 1

the span, and sometimes battered, according to the judgment of the engineer. In the present case, allowance will be made for a batter of ½ inch per foot in each abutment; then, the distance between the faces will increase at the rate

of $\frac{1}{2} + \frac{1}{2} = 1$ inch for every foot above the base of the rail, as far as the bottom of the bridge seat or coping. Bridge seat stones are usually made from 12 inches thick for short spans to 24 inches, and in some cases more, for long spans. In the present case, a thickness of 16 inches will be sufficient. Allowing 1 inch for the thickness of the sole plate and 1 inch for the bedplate makes the distance from the underneath clearance line (the bottom of the **I** beam) to the bottom of the bridge seat $16 + 1 + 1 = 18$ inches, and the distance from the base of the rail to the bottom of the bridge seat 22 feet − 1 foot 6 inches = 20 feet 6 inches. Then, as the distance between the abutments increases 1 inch for each foot, it will increase $20\frac{1}{2}$ inches, or 1 foot $8\frac{1}{2}$ inches, in 20 feet 6 inches, and the distance between the neat lines under the bridge seat will be 28 feet + 1 foot $8\frac{1}{2}$ inches = 29 feet $8\frac{1}{2}$ inches, as represented in Fig. 1, which is the **bridge-seat plan**, that is, the drawing of the bridge seat. [It will be noticed that here the word *plan* is used in the sense of *drawing* or *drawings*, not in the sense of a top view. In reality, the drawings in Fig. 1 show both a top view (*a*) and a cross-section (*b*).]

4. For a span of this length, the edge of the bedplate should not come closer than 3 or 4 inches to the neat line, and should not be set much farther back than this, as it lengthens the span. In the present case, it will be set $4\frac{3}{4}$ inches back at each abutment, making the clear distance between bedplates 29 feet $8\frac{1}{2}$ inches + $4\frac{3}{4}$ inches + $4\frac{3}{4}$ inches = 30 feet 6 inches. Bedplates are seldom made less than 12 inches in length; this is long enough for this span, and makes the total length of **I** beams 30 feet 6 inches + 1 foot + 1 foot = 32 feet 6 inches, and the distance center to center of bedplates (the span) 31 feet 6 inches, or 378 inches. The parapets are usually set 3 inches from the ends of the beams; that makes them, in this case, 33 feet apart.

5. Depth and Spacing of Beams.—As this span is less than 35 feet, rolled beams will be used. According to *B. S.*, Art. **92**, they cannot be less than $378 \div 30 = 12.6$ inches in

GENERAL DATA

For bridge over *Delaware, Lackawanna, & Western Railroad*

at *Elmhurst, Pennsylvania*

Length and general dimensions *To span two tracks 13 feet center to center*

Skew or angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *20 feet clear width. No trusses*

Floor system *One layer of 3-inch oak plank on nailing pieces and steel beams*

Number and location of tracks *No tracks*

Loading *Art. 98 (3) Bridge Specifications*

Description of abutments *Cement-concrete abutments*

Distance from floor to clearance line *Not more than 3 feet*

" " " " high water

" " " " low water

" " " " river bottom

Character of river bottom

Usual season for floods

Name of nearest railroad station *Elmhurst, Pennsylvania, D., L. and W. R. R.*

Distance to nearest railroad station *2 miles*

Time limit *6 months from date of award of contract*

Name of Engineer *International Textbook Company*

Address of Engineer *Scranton, Pennsylvania*

Remarks *To have a tight board fence at each side at least 5 feet 6 inches above the top of the floor*

depth. Table XIV* shows that the next depth of beam is 15 inches. In finding the spacing of beams, it is first necessary to determine the distance between the outside beams. In the present case, a roadway of 20 feet between wheel-guards is required; the fence will be placed 6 inches outside of the inner edges of the wheel-guards; then, the clear distance between the fences will be 21 feet. A tight board fence is specified to prevent injury to the traffic from cinders and sparks from the locomotives that will pass underneath.

Fig. 2 shows the cross-section of a good style of fence and connection to the beams; the posts are placed 5 or 6 feet apart. The outside beam is usually a channel, prefer-

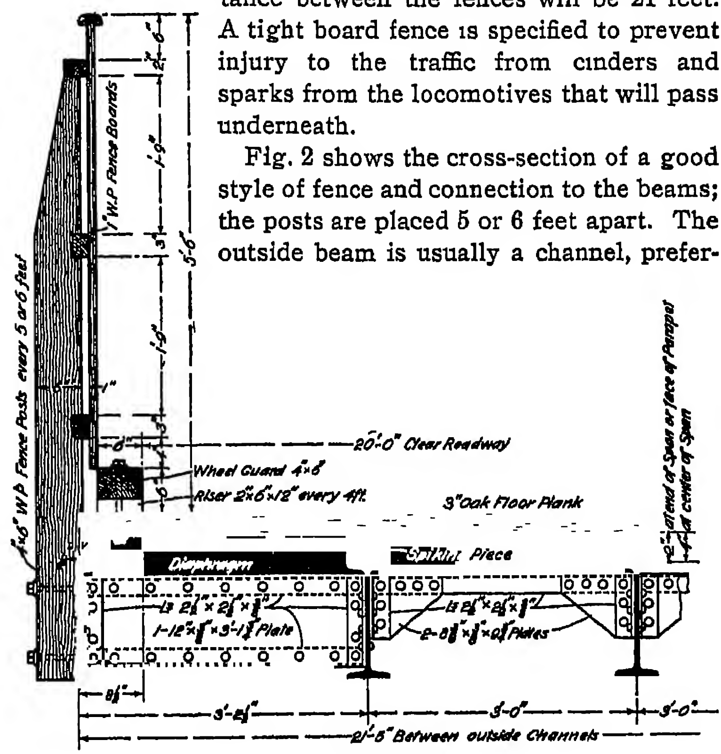


FIG. 2

ably of the same depth as the I beams. To assist in keeping the fence in place and preventing it from being blown over, the channel may be connected to the next beam, at intervals of about 10 feet, by means of the diaphragm shown in Fig. 2.

*All tables referred to in this Section are found in *Bridge Tables*, unless otherwise stated.

This may be made of the lightest material allowed—in this case, as the bridge is over a railroad, $\frac{3}{8}$ inch thick (*B. S.*, Art. **112**). With the arrangement represented in Fig. 2, the face of the channel is $8\frac{1}{2}$ inches outside of the edge of the wheel-guard, making the channels 20 feet + $8\frac{1}{2}$ inches + $8\frac{1}{2}$ inches = 21 feet 5 inches from face to face. As there will be one layer of floor planks 3 inches thick, the beams cannot be more than 3 feet center to center (*B. S*, Art. **123**). Five spaces at 3 feet makes 15 feet, leaving 21 feet 5 inches − 15 feet = 6 feet 5 inches, to be made up in the sides of the bridge. This requires the face of each channel to be placed one-half of 6 feet 5 inches, or 3 feet $2\frac{1}{2}$ inches, from the center of the first beam; if a spiking piece 4 inches wide is used on the channel, its center will be just 3 feet from the center of the next beam. This spacing of beams will satisfy all conditions.

6. Live Load.—According to *B. S.*, Arts. **98** (3) and **110**, the live load should be *either* 80 pounds per square foot *or* a steam road roller acting on each beam as two concentrated loads of 5,000 pounds, 11 feet apart. For the former, as the beams are 3 feet apart, the load per linear foot is $3 \times 80 = 240$ pounds The maximum bending moment occurs at the center, and, as the span is 31.5 feet, is

$$\frac{240 \times 31.5 \times 31.5}{8} = 29{,}770 \text{ foot-pounds}$$

The maximum bending moment due to the two concentrated loads occurs under one of the loads when that load and the center of gravity of the two loads are equidistant from the center of the beam, and, as explained in *Stresses in Bridge Trusses*, Part 4, is equal to $\frac{10{,}000 \times 13 \times 13}{31.5} = 53{,}650$ foot-pounds at 2.75 feet from the center of the span. As the bending moment due to the road roller is the greater, it is unnecessary to further consider that due to the uniform load. The shear will not be considered in this example, as it has been found in practice that, in all ordinary cases in bridge work, an I beam or channel that is strong enough to resist the bending moment is also strong enough to resist the shear

The bending moment on the channel may be taken equal to one-half that on the I beams, or 53,650 ÷ 2 = 26,820 foot-pounds, as the load on the channel is practically one-half the load on the beam.

7. Dead Load.—As 4.5 pounds per board foot, the weight stated in *B. S.*, Art. **97**, is rather high for the timber used in bridge floors, we shall assume it to include the weight of the spikes that hold the floor down, and the bolts that hold the spiking pieces to the beams. As the floor plank is 3 inches thick, its weight is 3 × 4.5 = 13 5 pounds per square foot, or, since the beams are 3 feet apart, 3 × 13.5 = 40 5 pound per linear foot of beam. The floor plank will be fastened down by spiking it to wooden nailing pieces 2 to 6 inches thick that are bolted to the tops of the I beams (see *B. S.*, Art. **122**). In order to prevent water from standing on the floor, as it would do if the floor were level, it is customary to make the floor a little higher at the center of the span than at the ends, this is done by varying the depth of the nailing pieces. In the present case, the nailing pieces will be made 2 inches deep at the ends of the span and 4 inches deep at the center. As the top of the nailing piece will be curved, its average depth will be about $3\frac{1}{2}$ inches. Its width should be at least equal to the flange of the beam; it will be assumed that 6 inches is sufficient. The average cross-section is then $3\frac{1}{2}$ in. × 6 in., equivalent to $1\frac{3}{4}$ board feet; this makes the weight per linear foot 1.75 × 4.5 = 7.9 pounds. The total weight of timber supported by each beam is, then, very nearly 40.5 + 7.9 = 48.4 pounds per linear foot. As the maximum bending moment due to the live load occurs 2.75 feet from the center, or 13 feet from the end of the span, it is necessary to find the dead-load bending moment at that point. For the floor plank and nailing pieces, it is

$$\frac{48.4 \times 31.5}{2} \times 13 - (48.4 \times 13) \times \tfrac{13}{2} = 5{,}820 \text{ foot-pounds}$$

for each beam, and approximately one-half of this, or 2,910 foot-pounds, for each channel.

8. There is still to be considered the bending moment due to the weight of the diaphragms represented in Fig. 2. The distance from the center of the I beam to the back of the channel is 3 feet $2\frac{1}{2}$ inches; the diaphragm will be about 3 feet $1\frac{1}{8}$ inches long, and, if 15-inch beams are used, about 12 inches deep. Then, as the weight of a $12'' \times \frac{3}{8}''$ plate is 15.3 pounds per linear foot, the weight of 3 feet $1\frac{1}{2}$ inches is $15.3 \times 3.125 = 47.8$ pounds. The total length of angle is 3 feet $1\frac{1}{2}$ inches + 3 feet $1\frac{1}{2}$ inches + 12 inches + 12 inches = 8 feet 3 inches, or 8.25 feet. As the weight of a $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ angle is 5.9 pounds per linear foot, the total weight of angles is $5.9 \times 8.25 = 48.7$ pounds. In Fig. 2, it may be seen that there are thirty $\frac{5}{8}$-inch rivets in one diaphragm, and the

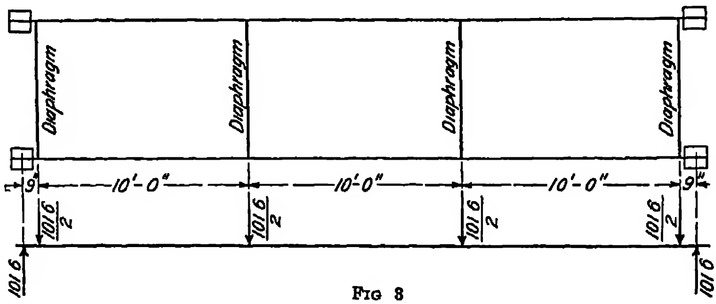


FIG 3

weight of their heads must be found. Table XXI gives 8.5 pounds for the weight of one hundred rivet heads for $\frac{5}{8}$-inch rivets; then, the weight of sixty rivet heads will be $\frac{60}{100} \times 8.5 = 5.10$ pounds. The weight of one diaphragm is, therefore, $47.8 + 48.7 + 5.1 = 101.6$ pounds. Using four of these placed symmetrically on the span at distances of about 10 feet, as shown in Fig. 3, the bending moment on the I beam and on the channel due to them is, since half of each goes to one beam,

$$101.6 \times 10.75 - \frac{101.6}{2} \times 10 = 580 \text{ foot-pounds}$$

The bending moments due to the weight of the beams and channels cannot be found until their weights are known. It is well to assume some value, however, and, if necessary,

correct it later An experienced designer will come very close the first time. It has been shown that the depth cannot be less than 15 inches; we shall use the weight of the lightest 15-inch beam and channel, 42 and 33 pounds per linear foot, respectively, as given in Tables XIII and XIV. For the I beams, the bending moment at the point of maximum moment, 2.75 feet from the center, is

$$\frac{42 \times 31.5}{2} \times 13 - (42 \times 13) \times \tfrac{13}{2} = 5{,}050 \text{ foot-pounds}$$

And for the channel,

$$\frac{33 \times 31.5}{2} \times 13 - (33 \times 13) \times \tfrac{13}{2} = 3{,}970 \text{ foot-pounds}$$

The guard timber and fence will be found to weigh very nearly 45 pounds per linear foot, and, as this weight is almost all carried by the channel, the moment on the channel due to it is

$$\frac{45 \times 31.5}{2} \times 13 - 45 \times 13 \times \tfrac{13}{2} = 5{,}410 \text{ foot-pounds}$$

The total maximum bending moment on an I beam is, then, 53,650 + 5,820 + 580 + 5,050 = 65,100 foot-pounds, or 781,200 inch-pounds.

The total maximum bending moment on a channel is 26,820 + 2,910 + 580 + 3,970 + 5,410 = 39,690 foot-pounds, or 476,300 inch-pounds.

9. Allowable Working Stress.—The allowable intensity of stress given in *B. S.*, Art. **103**, for the compression flange is $20{,}000 - 200 \times \frac{l}{w}$. It is not considered good practice to assume that a wooden floor gives lateral support to the top flanges of the I beams; so, if no bracing is placed between the beams, the unsupported length will be $31.5 \times 12 = 378$ inches. Suppose that a 15-inch beam were used, then, as the flange is about 5.5 inches wide, the ratio $\frac{l}{w}$ would be $\frac{378}{5.5} = 68.7$, and the allowable intensity of stress would be $20{,}000 - 200 \times 68.7 = 6{,}260$ pounds per square inch. If, however, small struts are placed between the beams near the top flanges, in the same relative positions

as the diaphragms already referred to, the unsupported lengths may be taken as 10 feet, or 120 inches, and the allowable intensity of stress for the I beam will then be

$$20{,}000 - 200 \times \frac{120}{5.5} = 15{,}640 \text{ pounds}$$

In general, when the ratio $\frac{l}{w}$ exceeds 40, the top flanges should be supported laterally.

As the bending moment is 781,200 inch-pounds, the required value of the section modulus is 781,200 ÷ 15,640 = 49.95 Consulting Table XIV, it is found that the lightest 15-inch beam—that is, a 15-inch 42-pound beam—has a section modulus of 58.9. This beam, therefore, can and will be used.

10. Assuming that the lightest 15-inch channel is used, for which the flange is 3.4 inches wide, the allowable intensity of stress is

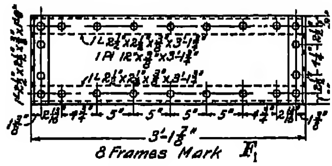
$$20{,}000 - 200 \times \frac{120}{3.4} = 12{,}940 \text{ pounds per square inch}$$

Then, as the maximum bending moment is 476,300 inch-pounds, the required value of the section modulus is 476,300 ÷ 12,940 = 36 81. Consulting Table XIII, it is found that a 15-inch 33-pound channel has a section modulus of 41.7, and so this will be used.

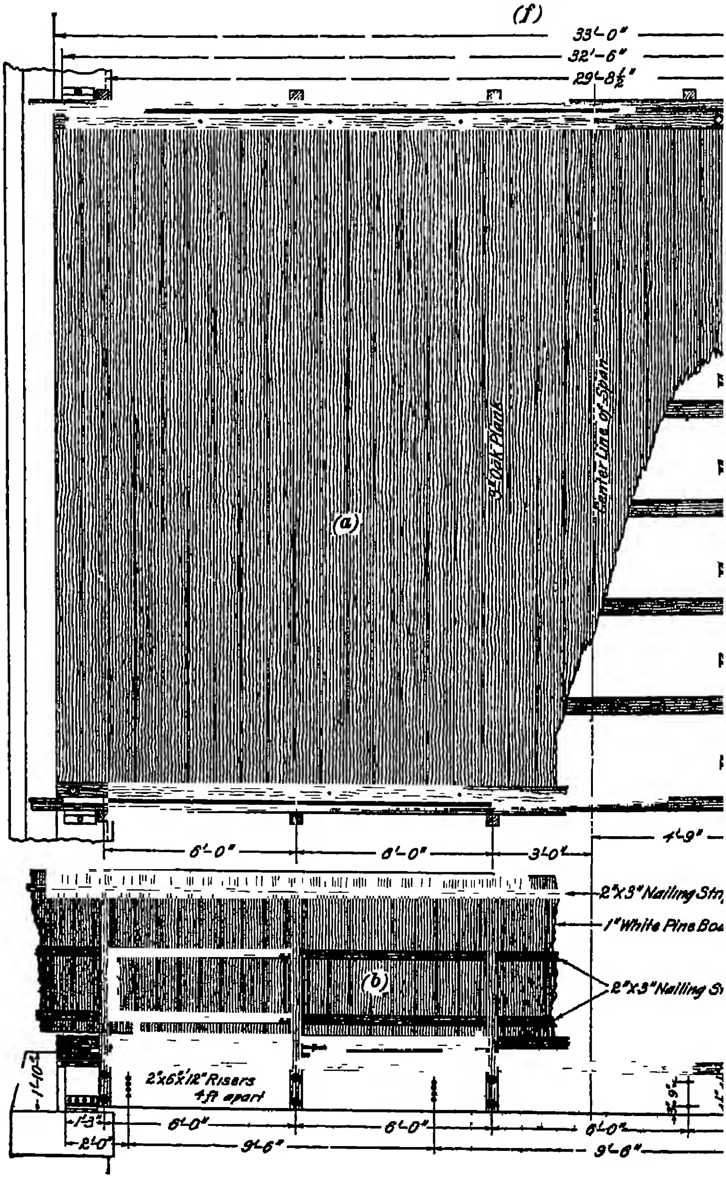
11. The bracing angles will add a little to the dead load on the I beams; but, as the section modulus of the 15-inch 42-pound I beam is larger than required, it is not necessary to consider the effect of those angles.

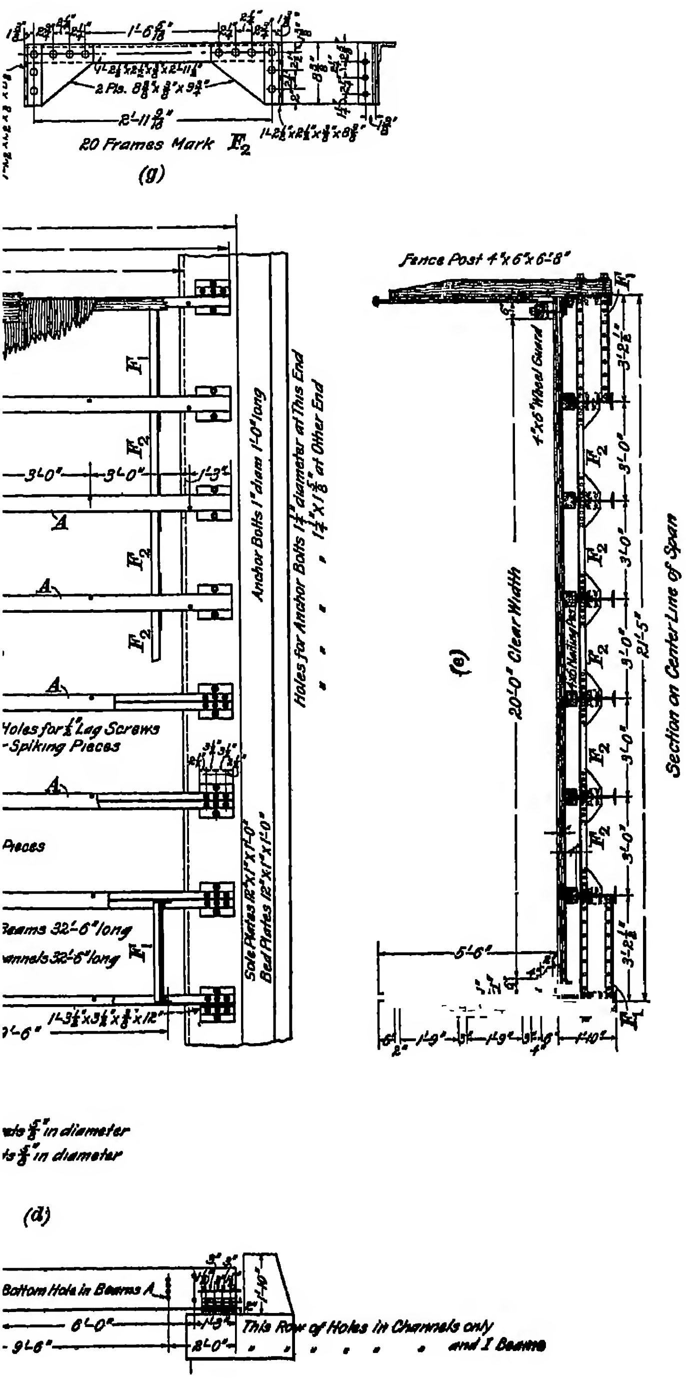
NOTE —In case the required value of the section modulus had come out larger than that corresponding to the assumed size of the beam, it would have been necessary to revise the design Suppose that it had come out 86 0 Then, if a 15-inch beam were used, it would be necessary to use a 15-inch 70-pound beam, for which the section modulus is 88 5, the same strength could be had, however, by using an 18-inch 55-pound beam, for which the section modulus is 88.4, thereby getting the same strength with a lighter beam. It would then be necessary to recompute the bending moment and allowable intensity of stress for the 18-inch beam

12. Depth of Floor.—The distance from the clearance line and from the top of the bridge seat to the top of the


3'-1 7/8"
8 Frames Mark F1

(f)


33'-0"
32'-6"
29'-8 1/2"
3" Oak Plank
Center Line of Span
(a)
4'-9"
6'-0"
6'-0"
3'-0"
2"X3" Nailing Str
1" White Pine Boa
(b)
2"X3" Nailing S
2"x6"x12" Risers 4 ft apart
1'-10"
1'-3"
2'-0"
9'-6"
9'-6"



20 Frames Mark F_2
(g)
Fence Post 4"x6"x6'-8"
4"x6" Wheel Guard
20'-0" Clear Width
(e)
21'-5"
Section on Center Line of Span
Anchor Bolts 1" diam 1'-0" long
Holes for Anchor Bolts $1\frac{1}{4}$" diameter at This End
$1\frac{1}{4}$"x$1\frac{5}{8}$" at Other End
Sole Plates 12"x1"x1'-0"
Bed Plates 12"x1"x1'-0"
Holes for $\frac{1}{2}$" Lag Screws
Spiking Pieces
Pieces
Beams 32'-6" long
Channels 32'-6" long
(d)
Bottom Hole in Beams A
This Row of Holes in Channels only
and I Beams

floor can now be found. The vertical distances at the center of the span are 3 inches of plank, 4 inches of nailing strip. and 15 inches for the I beams and channels, making 22 inches for the depth of floor at the center, as represented in Fig. 1 (*b*). In Art. **3**, 2 inches was allowed for sole plates and bedplates under the beams; the bridge seat is, therefore, 24 inches below the floor at the center of the span, and, since the nailing pieces are 2 inches deep at the ends, the bridge seat is 22 inches below the floor at the ends. If the tops of the parapets *a*, *a*, Fig. 1 (*b*), are made level with the floor, they must be 22 inches high.

13. General Plan or Detail Drawings.—Fig. 4 is a detail drawing of the bridge that has just been designed, and gives all the information necessary for its manufacture. It is customary, in drawing the plan and elevation, to show a portion of the floor and fence in its finished condition, as at (*a*) and (*b*), and the remainder with the floor and fence removed, as at (*c*) and (*d*). One end of a channel and several beams are usually shown with the top flange cut away, in order to show the detail of the connection of the beams to the sole plates. The diaphragms, sometimes called **frames**, are marked F_1, and the cross-struts, which also are called frames, are marked F_2. Frames of both kinds are shown to a larger scale at (*f*) and (*g*).

The holes for the anchor bolts at one end are made circular and ¼ inch larger in diameter than the bolts; at the other end, they are made 1¼ inches wide and 1⅝ inches long, allowing ⅜ inch for expansion and contraction.

In a bridge of this size, the diaphragms or frames are sometimes bolted instead of riveted to the beams, to avoid the expense of setting up a riveting plant for so small a number of rivets.

DESIGN OF AN I-BEAM RAILROAD BRIDGE

14. Data.—An I-beam railroad bridge will now be designed from the data given on page 13.

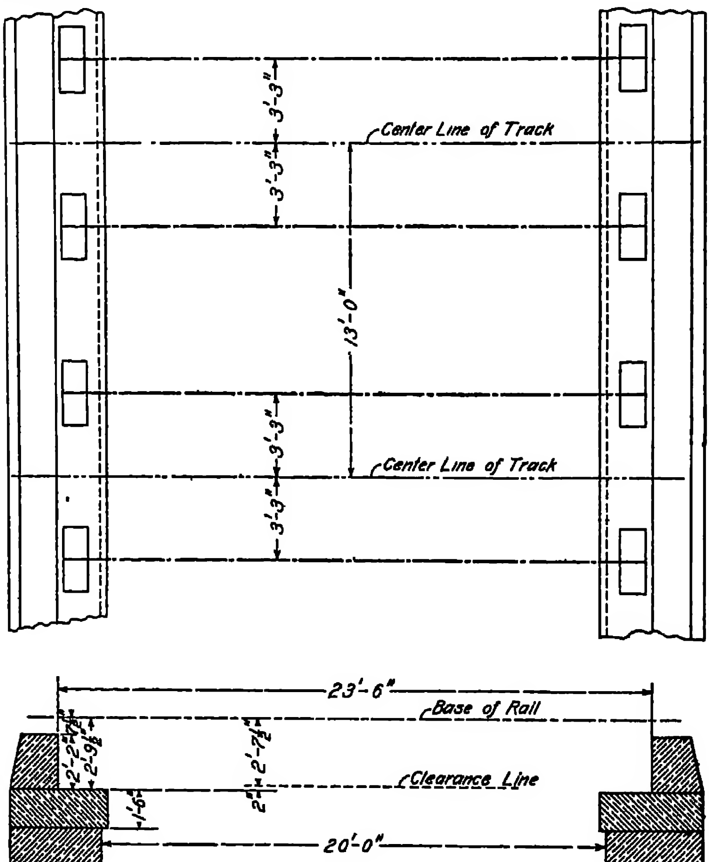


FIG. 5

15. Determination of Span.—The distance between neat lines under bridge seats is 20 feet. If the bearing

GENERAL DATA

For bridge over *French Creek*

at *Redlands, California*

Length and general dimensions *20 feet between neat lines under bridge seats*

Skew or angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *No trusses*

Floor system *Standard tie-floor on steel stringers or I beams*

Number and location of tracks *2 tracks 13 feet center to center*

Loading *Cooper's E50, as represented in Bridge Specifications, Art. 24*

Description of abutments *Cement concrete*

Distance from floor to clearance line *Not greater than distance to high water*

Distance from floor to high water *8 feet*

" " " " low water *13 feet*

" " " " river bottom *15 feet*

Character of river bottom *2 feet gravel, 3 feet shale, then solid rock 20 feet below base of rail*

Usual season for floods *April and May*

Name of nearest railroad station *Redlands, California*

Distance to nearest railroad station *5 miles*

Time limit *90 days*

Name of Engineer *Henry Jones*

Address *Berkeley, California*

Remarks

plates are made 12 inches wide, and set so that their edges are 6 inches back of the neat line, the total length of the beams will be 20 feet + 1 foot 6 inches + 1 foot 6 inches = 23 feet; and the distance center to center of bearings (the span), 1 foot less, or 22 feet. If 3 inches is left at each end, the parapets will be 23 feet 6 inches apart. The bridge seats will be made 1 foot 6 inches thick. The bridge-seat plan is represented in Fig. 5. The distance from the base of the rail to the top of the bridge seat cannot be found until after the bridge is designed. The top of the parapet is made $7\frac{1}{2}$ inches below the base of the rail.

16. Depth and Spacing of Beams.—The depth and the spacing of beams depend on the maximum bending moment and required value of the section modulus. It is considered better practice to place two beams under each rail than one; where necessary, three are used. The beams under each rail are bolted together and made to act as one by means of cast-iron separators or spacers, as illustrated in Table XV; these are located about 5 or 6 feet apart along the beams, but are not to be assumed as supporting the top flanges laterally. Lateral support is furnished by lateral bracing so arranged that the ratio of unsupported length to width of flange shall not exceed 12 (see *B. S.*, Art. **87**).

17. Live Load.—The live load is represented in Fig. 3 of *B. S.* By applying the conditions for maximum moment, it is found that it occurs when there are four driving axles on the span, under the second (or third) driver when that driver is 1.25 feet from the center of the span, and is equal to

$$\frac{4 \times 50{,}000 \times 9.75 \times 9.75}{22} - 50{,}000 \times 5 = 614{,}200 \text{ foot-pounds.}$$

18. Impact and Vibration.—The formula for impact and vibration is $I = \frac{300}{L + 300} \times S$ (*B. S.*, Art. **25**). In this case, as the moment is required, the value of the moment found in the last article should be substituted for S; and, as

the entire span must be loaded in order to produce the maximum moment, $L = 22$. Therefore,

$$I = \frac{300}{22 + 300} \times 614{,}200 = 572{,}200 \text{ foot-pounds}$$

19. Dead Load.—The weight w per linear foot of I-beam bridges of this class is given very closely by the formula $w = 25\,l$ (*B. S.*, Art. **242**). In this case, $l = 22$, and, therefore, $w = 25 \times 22 = 550$ pounds per linear foot. The weight of track can be taken as 400 pounds per linear foot (*B. S.*, Art. **23**). The maximum live-load moment occurs very near the center of the span, and it will be sufficiently accurate to find the dead-load moment at the center, which is

$$\frac{(550 + 400) \times 22 \times 22}{8} = 57{,}500 \text{ foot-pounds}$$

20. Wind Load.—It is unnecessary to compute the stresses in the laterals of I-beam bridges If the smallest-sized angle allowed is used—in this case, $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. —it is sufficiently strong to resist any wind stresses. According to *B. S.*, Art. **27**, the wind pressure on the train is 300 pounds per linear foot, applied 7 feet above the top of the rail. Ordinarily, the lateral system will be about 2 feet below the top of the rail, or about 9 feet below the center of wind pressure. Then, as the beams will be placed 6 feet 6 inches center to center, the additional load on the leeward beams (as explained in *Stresses in Bridge Trusses*, Part 5) will be $\frac{300 \times 9}{6.5} = 415$ pounds per linear foot, and the bending moment at the center due to it will be

$$\frac{415 \times 22 \times 22}{8} = 25{,}100 \text{ foot-pounds}$$

As this is so small, it is sometimes neglected. In the present case, it is less than 2 per cent. of the total moment, but, there being no reason why it should not be considered, it will be taken into account.

21. Total Moment.—The total moment is equal to the sum of the several moments just found, and is as follows:

614,200 + 572,200 + 57,500 + 25,100 = 1,269,000 foot-pounds = 15,228,000 inch-pounds.

22. Section Modulus.—As it is required that the flanges shall be supported laterally, the full value of 16,000 pounds per square inch can be used for the allowable intensity of stress, since the ratio of width to unsupported length will be less than 20 (see *B. S.*, Art. **29**). Then, the required value of section modulus is 15,228,000 ÷ 16,000 = 951.75.

If four beams are used (two under each rail), the required value of the section modulus for each will be 951.75 ÷ 4 = 237.94 Consulting Table XIV, it is found that the heaviest I beam made has a section modulus of 198.4, which is not enough. Therefore, more beams must be used. If six beams are used (three under each rail), the required value of the section modulus for each will be 951 75 ÷ 6 = 158.62. Consulting Table XIV, it is found that a 20-inch 95-pound I beam has a section modulus of 160.7, and a 24-inch 80-pound I beam has a section modulus of 174. If it were necessary to keep the distance from the base of the rail to the underneath clearance line as small as possible, the 20-inch beams would be used. In the present case, as there are practically no restrictions, the 24-inch beams will be used, as they are lighter and therefore more economical. On account of their larger value of section modulus, the 24-inch beams are also stronger than the 20-inch beams.

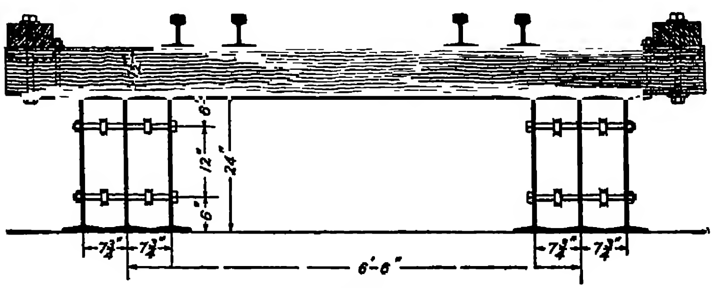
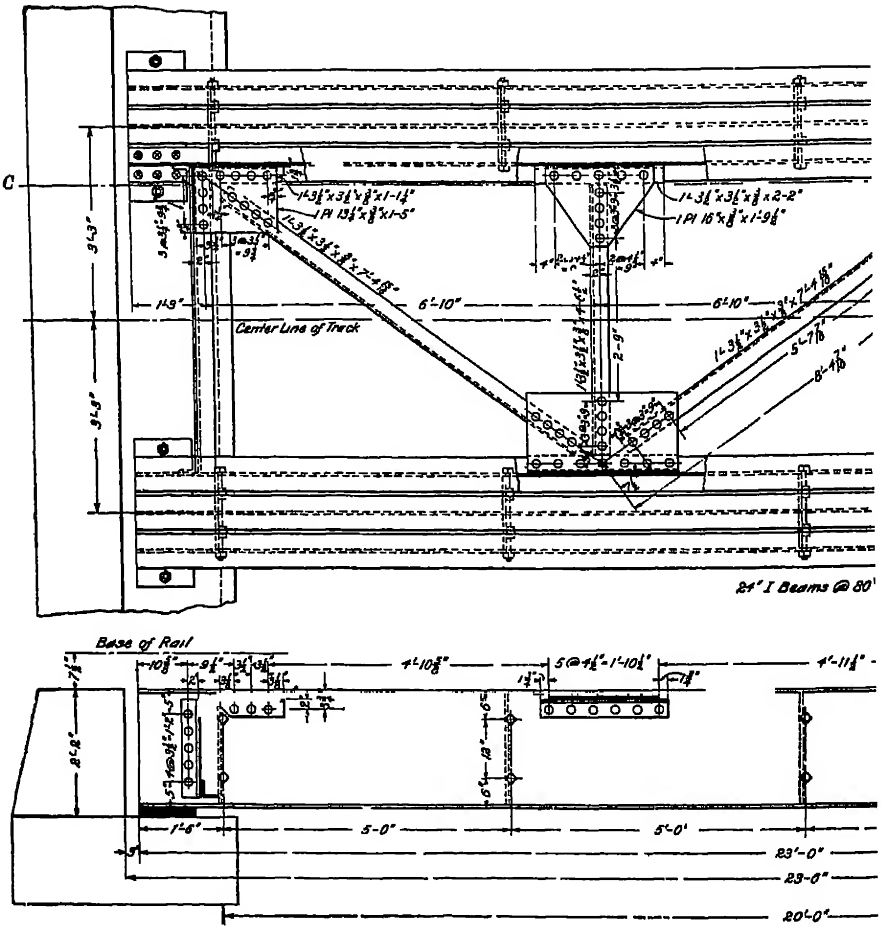
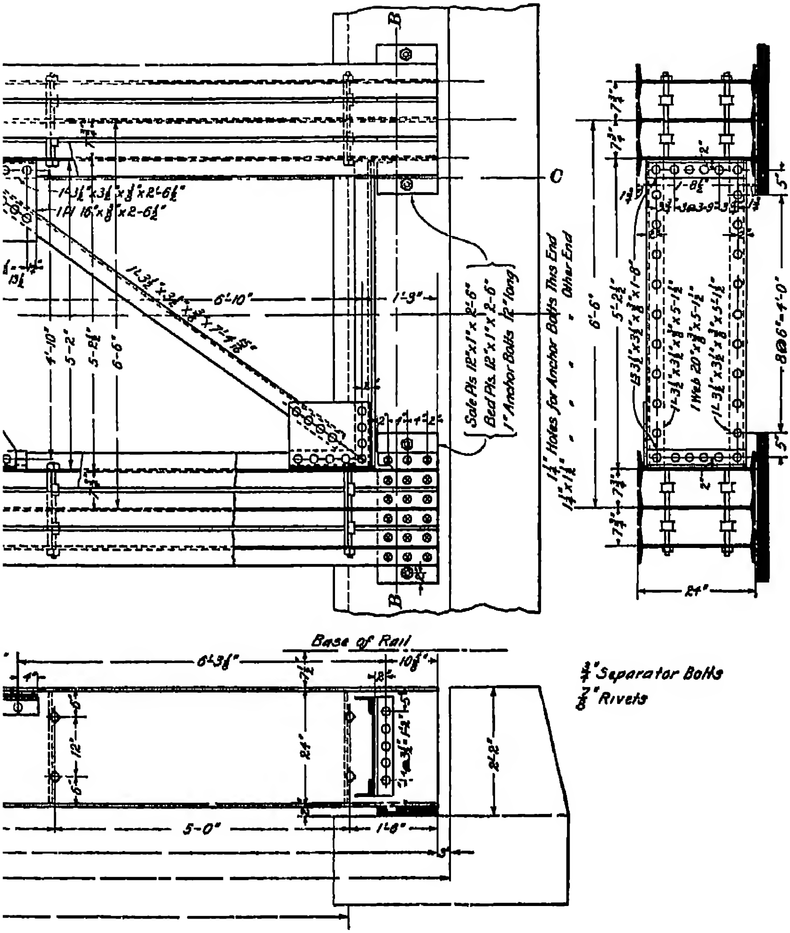


FIG 6

23. Depth of Bridge.—In *B. S.*, Art. **48**, it is stated that standard ties are framed to 7½ inches in depth over



Cross Section on



Cross Section on B-B

stringers and girders 6 feet 6 inches center to center. Then, the distance from the base of rail to the underneath clearance line, as represented in cross-section in Fig. 6, is $7\frac{1}{2}$ inches + 24 inches = 2 feet $7\frac{1}{2}$ inches. Allowing 1 inch for the sole plate and 1 inch for the bedplate gives 2 feet $9\frac{1}{2}$ inches from the base of the rail to the top of the bridge seat, as represented in Fig. 5

24. Plan.—Fig. 7 is a detail drawing of the bridge that has just been designed, and shows the customary method of arranging the lateral system. The lateral angles are connected to plates that are riveted to angles attached to the inside beams. The lateral truss is placed as high as is possible without interfering with the top flanges of the beams. The two sets of I beams are connected near the ends by diaphragms or frames, and at the panel points of the lateral truss by means of single angles. It is customary to show the top view of the I beams, and to consider a portion of the top flange removed at each lateral connection and at the end of one set of beams, in order to show more clearly the detail of the connection of the laterals to the beam and of the sole plates to the lower flanges.

For I-beam bridges longer than about 18 feet, three panels are used in the lateral systems; for shorter bridges, two panels are employed. The panels are usually made equal. In locating the rivets in the lateral connection plates, great care must be taken that the different angles connecting to the plate do not interfere with one another. It is well to lay out each connection plate on a large sheet full or half size and draw each angle in its proper position, leaving about $\frac{1}{4}$ inch clearance between the different angles that come close together.

The process of laying out the lateral system is frequently perplexing to a beginner, but is very simple after a little experience has been had. The different steps will be briefly outlined. The end frames are first located near the ends in such a position that they will not interfere with
bolts nor with the rivets that connect the sole

beams. This can be done by locating the frames about 1 inch from the sole plates. In Fig. 7, the backs of the angles that serve as flanges for the frames are 1 foot 1 inch from the ends of the I beams These angles are 3½ inches wide, and, according to Table XII, the gauge lines are 2 inches from the back edges. This makes the gauge lines of the end frames 1 foot 3 inches from the ends of the span and 23 feet − 1 foot 3 inches − 1 foot 3 inches = 20 feet 6 inches from each other. This distance is divided into three equal spaces of 6 feet 10 inches each, thus locating the gauge lines of the angles that act as struts between the beams at the intermediate points.

The three beams that form each side of the bridge are bolted together with their centers 7¾ inches apart, as given in Table XV, and each set of three is placed with its center 6 feet 6 inches from that of the other. This makes the inside beams 6 feet 6 inches − 7¾ inches − 7¾ inches = 5 feet 2½ inches apart. Since the webs of these beams are ½ inch in thickness (Table XIV), the clear distance between them is 5 feet 2 inches The lug or hitch angles that are used to connect the plates to the webs of the inside I beams are 3½ inches wide, with the gauge lines 2 inches from the webs. This makes these gauge lines 5 feet 2 inches − 2 inches − 2 inches = 4 feet 10 inches apart. The gauge lines of these hitch angles and those of the cross-frames or struts, located in the preceding paragraph, are commonly called **working lines** for the lateral trusses. They can be considered as the center lines of the chords and the panel points, respectively. The diagonal lines connecting the intersections of these gauge lines locate the diagonals of the lateral truss. In Fig. 7, they are taken as the gauge lines of the laterals. In some cases, the center of gravity of the angle is made to coincide with the diagonal between the working lines; in a bridge as small as that now under consideration, it matters little which method is used.

The next step is to find the length of the diagonal. Each of these lines, together with the working lines, forms a right

triangle whose two legs are 6 feet 10 inches and 4 feet 10 inches, respectively. Then, the length of the diagonal is

$$\sqrt{(6'\,10'')^2 + (4'\,10'')^2} = 8 \text{ feet } 4\tfrac{7}{16} \text{ inches}$$

The rivets at the ends of the laterals are next located by laying out the connection plates full or half size, as previously described.

In detailing bridge work, it is frequently necessary to find the length of the hypotenuse of a right triangle when the legs, commonly called the **coordinates** in this work, are known. The principle for finding the hypotenuse is simple, but the arithmetical work is laborious, especially when the coordinates are given to small fractions, such as sixteenths of an inch. To shorten this work, tables are commonly employed. There are several books on the market that contain tables giving the squares of distances that occur in feet, inches, and fractions of an inch. In using such tables, the squares of the coordinates are copied and added; the length corresponding to the square root of the sum is then taken directly from the table.

DESIGN OF A PLATE-GIRDER RAILROAD BRIDGE

25. Data.—As the next example of practical design will be taken a deck plate-girder railroad bridge, the data sheet for the construction of which is given on page 20.

26. Determination of Span.—It is first necessary to determine the location of the abutments. It is required that the clear distance between them at the level of the curb be made 50 feet. There is one electric-car track in the center of the street, and it may be assumed that the top of the rail is level with the top of the curb. According to *B. S.*, Art. **94**, the lowest line of overhead bracing in through bridges for street railways shall not be lower than 15 feet from the top of the rail. This condition applies equally well to the underneath clearance of bridges over street-railroad tracks. In the present case, the underneath clearance line of the bridge will be placed 15 feet above the top

GENERAL DATA

For bridge over *Sumner Street*

at *Cincinnati, Ohio*

Length and general dimensions *One span over street 50 feet wide at the level of the curb, with one electric-car track at the center*

Skew or angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *No trusses. Space deck girders according to Bridge Specifications, Art. 16*

Floor system *Standard-tie floor. See Bridge Specifications, Art. 48*

Number and location of tracks *2 tracks 13 feet center to center*

Loading *Cooper's E50, as represented in Bridge Specifications, Art. 24*

Description of abutments *Granite abutments, front faces even with street lines*

Distance from floor to clearance line *Not more than 6 feet 6 inches*

Distance from floor to high water *No water*

" " " " low water " "

" " " " river bottom " "

Character of river bottom " "

Usual season for floods " "

Name of nearest railroad station *Cincinnati, Ohio*

Distance to nearest railroad station *3 miles*

Time limit *90 days*

Name of Engineer

Address of Engineer

Remarks *Above-described bridge is to replace the bridge at present in use. New abutments will be built by the Railroad Company. Contractor will furnish bridge-seat plan within 10 days from award of contract*

of the rail, and the top of the bridge seat will be made level with that line. Bridge seats for this length of span should be not less than 18 inches thick; using this thickness, the distance from the top of the rail to the neat line under the bridge seat will be 15 feet − 1 foot 6 inches, or 13 feet 6 inches. If the face of each abutment is battered $\frac{1}{2}$ inch per foot, the abutments will be $13\frac{1}{2}$ inches farther apart under the bridge seat than at the top of the curb, or, in this case, 50 feet + 1 foot $1\frac{1}{2}$ inches = 51 feet $1\frac{1}{2}$ inches. Bedplates for deck plate girders are usually made about 20 inches long for spans of 50 feet, and about 24 inches long for spans 75 feet long, with the front edge from 6 to 9 inches behind the neat line. For the span under consideration, the plates will be made 22 inches in length and set with the front edges $6\frac{1}{4}$ inches behind the neat line. The center of the bedplate is then 11 inches + $6\frac{1}{4}$ inches = $17\frac{1}{4}$ inches, behind at each end of the span; the span is 51 feet $1\frac{1}{2}$ inches + 1 foot $5\frac{1}{4}$ inches + 1 foot $5\frac{1}{4}$ inches = 54 feet; and the total length of the girder is 22 inches more than this, or 55 feet 10 inches. For a girder of this length, the parapets should not come nearer than 4 inches to the ends. In this case, they will be made 56 feet 6 inches apart.

27. Depth and Spacing of Girders.—Deck girders are usually made about one-ninth or one-tenth of the span in depth. When the distance from the base of the rail to the clearance line is specified, the depth of the girder must be chosen so as not to exceed it. The depth of the tie and the thickness of the flange plates will usually occupy about 1 foot of the depth. In the present case, as the specified distance is 6 feet 6 inches, the depth of the girder, or the width of the web, should not exceed 5 feet 6 inches. If the depth of the girder is made one-tenth of the span, it will be $54 \div 10 = 5.4$ feet. It is well, when possible, to use an even foot or half foot for the width of web; in this case, 5.5 feet, or 66 inches, will be used. The backs of the flange angles are usually placed $\frac{1}{4}$ inch farther apart, or, in this case, $66\frac{1}{4}$ inches. This allows for slight irregularities in the

edges of the web-plates, and prevents the edges from interfering with the flange plates As the span is less than 70 feet, the girders will be 6 feet 6 inches center to center (see *B. S.*, Art. **16**).

The bridge-seat plan is represented in Fig. 8. The distance from the base of the rail to the clearance line or the

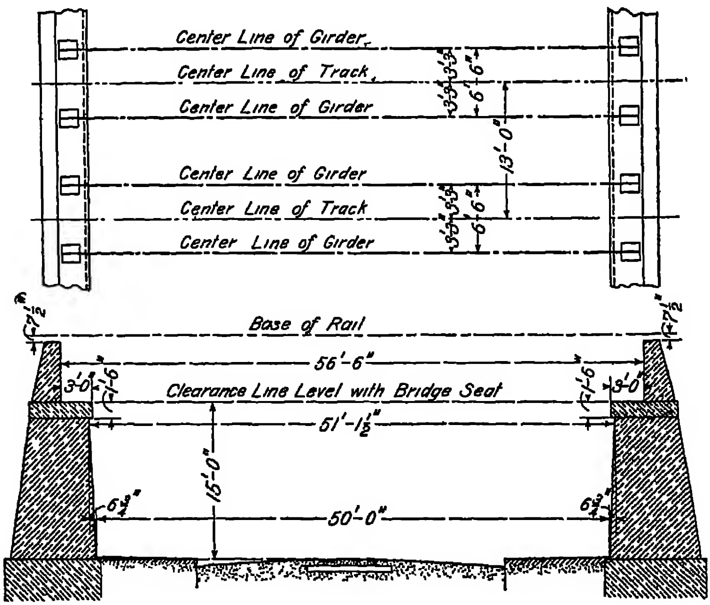


FIG 8

bridge seat is not given, as this distance cannot be determined until after the girders have been designed

28. Dead Load.—The formula for the weight per linear foot of a deck plate-girder railroad bridge for the specifications referred to in the data sheet, and for Cooper's E50 loading, is given in Art. **242**, *B. S.*, as

$$w = 500 + 8\,l = 500 + 8 \times 54 = 930 \text{ pounds, nearly}$$

The weight of the track will be taken as 400 pounds, making the total dead-weight $930 + 400 = 1{,}330$ pounds per linear foot for one track (two girders). As explained in *Design of Plate Girders*, Part 1, it is necessary to

calculate the moments and shears at several points along the girder. In the present case, they will be calculated at the center and at distances of 5, 10, 15, and 20 feet from the end of the span; besides, the shear at the end must be computed.

The dead-load moments, in foot-pounds, are as follows:

At the center,

$$\frac{1{,}330 \times 54 \times 54}{8} = 484{,}900$$

At 20 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 20 - \frac{1{,}330 \times 20 \times 20}{2} = 452{,}200$$

At 15 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 15 - \frac{1{,}330 \times 15 \times 15}{2} = 389{,}000$$

At 10 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 10 - \frac{1{,}330 \times 10 \times 10}{2} = 292{,}600$$

At 5 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 5 - \frac{1{,}330 \times 5 \times 5}{2} = 162{,}900$$

The dead-load shears, in pounds, are as follows:

At the end, $$\frac{1{,}330 \times 54}{2} = 35{,}910$$

At 5 feet from the end,

$$35{,}910 - 5 \times 1{,}330 = 29{,}260$$

At 10 feet from the end,

$$35{,}910 - 10 \times 1{,}330 = 22{,}610$$

At 15 feet from the end,

$$35{,}910 - 15 \times 1{,}330 = 15{,}960$$

At 20 feet from the end,

$$35{,}910 - 20 \times 1{,}330 = 9{,}310$$

At the center,

$$35{,}910 - 27 \times 1{,}330 = 0$$

29. Live Load.—The live load consists of Cooper's E50, represented in Fig. 8 of *B. S.* By applying the principles explained in *Stresses in Bridge Trusses*, Part 4, it is

found that the greatest moment occurs under the third driver of the second engine, when that driver is .127 foot to the left of the center. The value of this moment is 2,703,200 foot-pounds. The maximum moment at the center, in this case, occurs when the same driver is at the center, and is 2,702,500 foot-pounds. As will be seen, these two values are very nearly equal. In general, it may be stated that *for spans over 50 feet it is sufficiently accurate to use in design the greatest moment that can occur at the center of the span, neglecting the consideration that involves the location of the center of gravity of the loads.* The maximum live-load moments and shears are then as follows:

At the center (under third driver, second engine), the live-load moment, in foot-pounds, is 2,702,500.

At 20 feet from the end (under third driver, first engine), 2,563,000.

At 15 feet from the end (under second driver, first engine), 2,247,200.

At 10 feet from the end (under second driver, second engine), 1,703,700.

At 5 feet from the end (under first driver, second engine), 976,900.

At the end (first driver, second engine), the live-load shear, in pounds, is 228,900.

At 5 feet from the end (first driver, second engine), 195,400.

At 10 feet from the end (first driver, first engine), 163,100.

At 15 feet from the end (first driver, first engine), 130,900.

At 20 feet from the end (first driver, first engine), 101,600.

At the center (first driver, first engine), 65,200.

30. Impact and Vibration.—The formula for impact and vibration is $I = \frac{300}{L + 300} \times S$ (*B S.*, Art. **25**). In the present case, the allowance in terms of the moments and shears must be found. For the moments, it is necessary to load the entire span; then, $L = 54$, and

$$I = \tfrac{300}{354} \times M = .84746\, M$$

The allowances are:

LOCATION	MOMENT, IN FOOT-POUNDS
Center	$.84746 \times 2{,}702{,}500 = 2{,}290{,}300$
20 feet from the end	$.84746 \times 2{,}563{,}000 = 2{,}172{,}000$
15 feet from the end	$.84746 \times 2{,}247{,}200 = 1{,}904{,}400$
10 feet from the end	$.84746 \times 1{,}703{,}700 = 1{,}443{,}800$
5 feet from the end	$.84746 \times 976{,}900 = 827{,}900$

As the length of track that must be loaded to produce the greatest shears is different for different sections, the allowance for impact and vibration will be a different proportion of the maximum shear in each case. It has been found that the maximum shear at each section occurs when the first driver is at the section, for which position the first wheel is 8 feet beyond the section. The length of loaded track for sections within 8 feet of the end of the span is, therefore, 54 feet; for other sections it is equal to the distance of the section from the other end of the span plus 8 feet. The proportional allowances are as follows:

At the end and 5 feet from the end, the length L of loaded portion is 54; the proportional allowance, $I = .84746\ S$.

At 10 feet from the end, $L = 52$; $I = \frac{300}{352} S = .8523\ S$

At 15 feet from the end, $L = 47$; $I = \frac{300}{347} S = .8646\ S$

At 20 feet from the end, $L = 42$; $I = \frac{300}{342} S = .8772\ S$

At the center, $L = 35$; $I = \frac{300}{335} S = .8955\ S$

The allowances for shear are, therefore, as follows:

LOCATION	SHEAR, IN POUNDS
End	$.84746 \times 228{,}900 = 194{,}000$
5 feet from the end	$.84746 \times 195{,}400 = 165{,}600$
10 feet from the end	$.8523 \times 163{,}100 = 139{,}000$
15 feet from the end	$.8646 \times 130{,}900 = 113{,}200$
20 feet from the end	$.8772 \times 101{,}600 = 89{,}100$
Center	$.8955 \times 65{,}200 = 58{,}400$

31. Wind Pressure.—In this article, we shall consider only the increase in moments and shears caused in the leeward girder by the overturning effect of the wind; this increase is calculated by the formula $w = \frac{Ph}{b}$, in which P is the wind

pressure per linear foot of train, and w is the vertical load per linear foot of girder, due to the wind pressure (see *Stresses in Bridge Trusses*, Part 5). The center of the wind pressure is 7 feet above the top of the rail (*B. S*, Art. 27), and the top lateral bracing is generally about 2 feet below the top of the rail; then, h, the distance from the center of wind pressure to the top lateral bracing, is $7 + 2 = 9$ feet; and b is 6.5 feet. Therefore,

$$w = \frac{300 \times 9}{6.5} = 415 \text{ pounds per linear foot}$$

The moments due to a uniform load of 1,330 pounds per linear foot have already been found, those due to a uniform load of 415 pounds may be found by multiplying the former moments by $\frac{415}{1330}$, or .31203. The results are as follows:

Location	Moment, in Foot-Pounds
Center	$.31203 \times 484{,}900 = 151{,}300$
20 feet from the end	$.31203 \times 452{,}200 = 141{,}100$
15 feet from the end	$.31203 \times 389{,}000 = 121{,}400$
10 feet from the end	$.31203 \times 292{,}600 = 91{,}300$
5 feet from the end	$.31203 \times 162{,}900 = 50{,}800$

As the wind pressure under consideration is that on a moving train, the shears will be found as for a moving load, that is, by loading the portion of the span on one side of a section. They are as follows:

Location	Shear, in Pounds
End	$\frac{415 \times 54}{2} = 11{,}200$
5 feet from the end	$\frac{415 \times 49 \times 49}{2 \times 54} = 9{,}200$
10 feet from the end	$\frac{415 \times 44 \times 44}{2 \times 54} = 7{,}400$
15 feet from the end	$\frac{415 \times 39 \times 39}{2 \times 54} = 5{,}800$
20 feet from the end	$\frac{415 \times 34 \times 34}{2 \times 54} = 4{,}400$
Center	$\frac{415 \times 27 \times 27}{2 \times 54} = 2{,}800$

32. Total Moments and Shears.—As the dead load, live load, impact and vibration, and wind pressure may act simultaneously, the total moments and shears may be found by adding the values that have been found for the different conditions. In doing so, however, it must be remembered that the moments and shears due to dead load, live load, and impact and vibration have been found for the load on the entire width of track, that is, on two girders, while those due to the wind pressure are the effects on one girder. Therefore, to find the total moment or shear at any section of one girder, that due to the wind pressure may be added to one-half the sum of those due to dead load, live load, and impact and vibration, as just found. The total moments and shears *on each girder* can now be found.

The total moments, in foot-pounds, at various positions on the girder are as follows:

At the center,

$$\frac{484{,}900 + 2{,}702{,}500 + 2{,}290{,}300}{2} + 151{,}300 = 2{,}890{,}200$$

At 20 feet from the end,

$$\frac{452{,}200 + 2{,}563{,}000 + 2{,}172{,}000}{2} + 141{,}100 = 2{,}734{,}700$$

At 15 feet from the end,

$$\frac{389{,}000 + 2{,}247{,}200 + 1{,}904{,}400}{2} + 121{,}400 = 2{,}391{,}700$$

At 10 feet from the end,

$$\frac{292{,}600 + 1{,}703{,}700 + 1{,}443{,}800}{2} + 91{,}300 = 1{,}811{,}400$$

At 5 feet from the end,

$$\frac{162{,}900 + 976{,}900 + 827{,}900}{2} + 50{,}800 = 1{,}034{,}700$$

The total shears, in pounds, at the different sections of the girder are:

At the end,

$$\frac{35{,}910 + 228{,}900 + 194{,}000}{2} + 11{,}200 = 240{,}600$$

At 5 feet from the end,

$$\frac{29,260 + 195,400 + 165,600}{2} + 9,200 = 204,300$$

At 10 feet from the end,

$$\frac{22,610 + 163,100 + 139,000}{2} + 7,400 = 169,800$$

At 15 feet from the end,

$$\frac{15,960 + 130,900 + 113,200}{2} + 5,800 = 135,800$$

At 20 feet from the end,

$$\frac{9,310 + 101,600 + 89,100}{2} + 4,400 = 104,400$$

At the center,

$$\frac{0 + 65,200 + 58,400}{2} + 2,800 = 64,600$$

33. Design of Web.—A $\frac{7}{16}$-inch web will be tried first. The gross section is $66 \times \frac{7}{16} = 28.875$ square inches. As the total shear at the end is 240,600 pounds, the intensity of the shearing stress is $240,600 \div 28.875 = 8,330$ pounds per square inch. Consulting Table XXXVI, finding the point on the curve for the $\frac{7}{16}$-inch web corresponding to an intensity of stress of 8,330 pounds per square inch, and looking horizontally to the right, it is found that the stiffeners must be spaced about 16 inches apart. According to *B. S*, Art. **55**, the spacing of stiffeners should be not less than one-third the depth of the web. In this case, therefore, the stiffeners should be not less than 22 inches apart. As the spacing given in Table XXXVI is 16 inches, it is necessary to try a thicker web A web $\frac{1}{2}$ inch thick will be tried next. The gross section is $66 \times \frac{1}{2} = 33$ square inches, and the intensity of shearing stress, $240,600 \div 33 = 7,290$ pounds per square inch. Consulting Table XXXVI, finding the point on the curve for the $\frac{1}{2}$-inch web corresponding to an intensity of stress of 7,290 pounds per square inch, and looking horizontally to the right, it is found that the stiffeners must be spaced 22 inches apart. As this thickness of web satisfies the conditions as far as stiffener spacing at the end is

concerned, the required spacing of stiffeners at other sections will next be found. The intensities of shearing stress are:

Location	Intensity of Shearing Stress, in Pounds per Square Inch
End	240,600 ÷ 33 = 7,290
5 feet from the end	204,300 ÷ 33 = 6,190
10 feet from the end	169,800 ÷ 33 = 5,150
15 feet from the end	135,800 ÷ 33 = 4,120
20 feet from the end	104,400 ÷ 33 = 3,160
Center of span	64,600 ÷ 33 = 1,960

34. Spacing of Stiffeners.—Consulting Table XXXVI, finding the points on the curve for $\frac{1}{2}$-inch web corresponding to the intensities just found, and looking horizontally to the right or left (whichever is nearer), the required spacings of stiffeners are found to be as follows:

Location	Spacing of Stiffeners, in Inches
End	22
5 feet from the end	27
10 feet from the end	32
15 feet from the end	38
20 feet from the end	46
Center of span	62

The stiffener spacing at other points on the girder may be found by interpolating between the values just found. These distances will not give the actual distances between the stiffeners, but simply the distances that must not be exceeded at the various sections. The actual distances between stiffeners depend on other details, such as rivet spacing, and are usually found by the detailer or draftsman when the plans are being made.

35. Pitch of Flange Rivets.—The required spacing of the rivets that connect the flanges to the web at any section is found by the following formula, given in *Design of Plate Girders*, Part 1:

$$p = \frac{K h_r}{V}$$

In the present case, the rivets are in double shear and in bearing on the $\frac{1}{2}$-inch web-plate. (They are also in bearing on the two flange angles, but it may be assumed that the thickness of the latter is greater than that of the web; it is found in practice that this is invariably true. For this reason, the bearing on the flange angles need not be considered.) The rivets used in deck plate-girder railroad bridges are $\frac{7}{8}$ inch in diameter for all spans. Those in the flanges are always shop-driven rivets and, according to *B. S.*, Art. 29, the values in Table XL will be used. Consulting Table XL, the value of one $\frac{7}{8}$-inch rivet in double shear is found to be 13,230 pounds, and in bearing on a plate $\frac{1}{2}$ inch thick, 9,630 pounds. The latter value, being the smaller, must be used in the formula. As the flanges have not yet been designed, it is not known what size of angles will be used, so that the value of h_r cannot be calculated. In actual practice, however, the designer soon learns what sizes of angles are generally used for spans of different lengths.

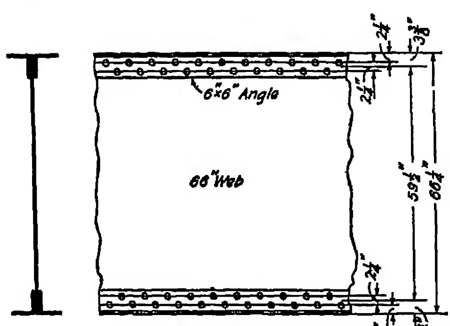


FIG. 9

In the type of bridge now under consideration, 6″ × 6″ angles are used for all spans up to about 70 feet, and 8 in. × 8 in. for longer spans In the present case, 6″ × 6″ angles will be used. Consulting Table XII, it is found that there will be two rows of rivets in each leg, the line midway between the two rows being $2\frac{1}{4} + \frac{2\frac{1}{4}}{2} = 3\frac{3}{8}$ inches from the back of the angle, as represented in Fig. 9. The distance between the backs of the angles in the flanges has already been found to be $66\frac{1}{4}$ inches. The distance h_r is, therefore, $66\frac{1}{4} - 3\frac{3}{8} - 3\frac{3}{8} = 59\frac{1}{2}$ inches. By substituting the proper values in the formula, and using the intensities of

shearing stress found in Art. **33**, the following pitches are obtained:

Location	Rivet Pitch, in Inches	
	Bottom Flange	*Top Flange*
End	$\frac{9{,}630 \times 59.5}{240{,}600} = 2.38$;	$2.38 \times .9 = 2.14$
5 feet from the end	$\frac{9{,}630 \times 59.5}{204{,}300} = 2.80$;	$2.80 \times .9 = 2.52$
10 feet from the end	$\frac{9{,}630 \times 59.5}{169{,}800} = 3.37$;	$3.37 \times .9 = 3.03$
15 feet from the end	$\frac{9{,}630 \times 59.5}{135{,}800} = 4.22$;	$4.22 \times .9 = 3.80$
20 feet from the end	$\frac{9{,}630 \times 59.5}{104{,}400} = 5.49$;	$5.49 \times .9 = 4.94$
Center	$\frac{9{,}630 \times 59.5}{64{,}600} = 8.87$;	$8.87 \times .9 = 7.98$

As a deck railroad bridge is under consideration, the pitch in the bottom flange, found from the formula $p = \frac{K h_r}{V}$, must be multiplied by .9 to get the required pitch in the top flange (see *B. S.*, Art. **57**). The pitch is sometimes made the same in both flanges, that found for the top flange in the manner just explained being used for the bottom as well. As in the case of stiffener spacing, the foregoing values will not represent the actual spacing of rivets at any section, but simply the values that must not be exceeded at the different sections. The spacing at other points may be found by interpolating between the given values. As the pitch at 20 feet from the end came out greater than that allowed in *B. S.*, Art. **57**, there was no necessity for computing the pitch at sections nearer the center.

36. Design of Flanges.—The required area of cross-section of the flanges at any section is found by means of the formula $A = \frac{M}{s h_g} - \frac{t h}{8}$ (*Design of Plate Girders*, Part 1). It is impossible to calculate the value of h_g, as the areas of the flanges are not yet known; for a first trial, however, h_g will be assumed equal to the depth h of web, the flanges

at the center of the span will be designed on this basis, using the bending moment at the center of the span, and the distance h_g between the centers of gravity of the trial flanges will be computed. With this corrected value for h_g, the areas of the flanges will again be found, and the necessary correction made. It will seldom be found necessary to change the cross-section of the flanges as found by assuming $h_g = h$. In the present case, $h = 66$ inches, $s = 16{,}000$ pounds, and $t = \frac{1}{2}$ inch. To apply the formula, the bending moment already found must be multiplied by 12, to reduce it to inch-pounds. For the first trial cross-section at the center of the span, we have, therefore,

$$\frac{2{,}890{,}200 \times 12}{66 \times 16{,}000} - \tfrac{1}{8} \times \tfrac{1}{2} \times 66 = 32.84 - 4.12$$
$$= 28.72 \text{ square inches}$$

This is the value for the gross area of the top flange and the net area of the bottom flange.

37. The actual choice of the sizes of angles and plates is wholly a matter of practice. The designer usually follows certain established rules and relies to a great extent on his experience. It is considered bad practice to use very small or thin angles and a large number of plates. It is also considered bad practice to make the entire flange section of angles; this is not economical, as the entire section of flange must be continued the whole length of the girder. In *B. S.*, Art. **59**, it is required that one-third to one-half the flange area shall be composed of angles. In the present case, that would require from $\frac{28.72}{3}$, or 9.57, to $\frac{28.72}{2}$, or 14.36, square inches in two angles, or, as there are two angles, 4 79 to 7.18 square inches in each angle. $6'' \times 6''$ angles with thicknesses from $\frac{1}{2}$ to $\frac{11}{16}$ inch are commonly used in the flanges of plate girders for railroad bridges up to about 70 feet in length; the flange plates are never narrower than the total width of the two flange angles together with the thickness of the web, nor thicker than the flange angles. In the present case, the following sections will be used:

Top Flange		Section, in Square Inches
Two angles 6 in. × 6 in. × $\frac{1}{2}$ in. @ 5.75	.	11.5
One plate 14 in. × $\frac{3}{8}$ in.	. . . .	5.25
One plate 14 in. × $\frac{7}{16}$ in.	. . . .	6.125
One plate 14 in. × $\frac{7}{16}$ in.	. . . .	6.125
Total gross area	. . . .	29.0

In *B. S.*, Art. **60**, it is required that, when plates of different thicknesses are used, they shall diminish in thickness outwards from the flange angles. An exception is sometimes made to this rule, and will be made in this case, when it is required that one plate shall extend the full length of the top flange. This is done to keep water and dirt from working down between the flange angles and the web, and to give the girder a better finish. A thin plate serves the purpose just as well as a thicker one, and is more economical, as all that part beyond its theoretical end is wasted, so far as necessary flange section is concerned.

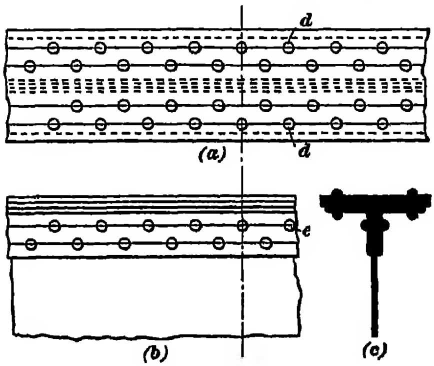


Fig 10

For the bottom flange it is necessary to deduct from the gross section the areas of cross-section of the rivet holes. This is most easily done by deducting from each angle and plate the holes that are in the plate. Fig. 10 shows the method of riveting flange angles to the web and the flange plates to the angles. Each rivet *d* in the horizontal leg of any angle is directly opposite one *e* in the vertical leg; this brings two rivets *d*, *d* directly opposite each other in each plate. There are then two holes to be deducted from each angle and two from each plate. The following sections will be used:

Bottom Flange		Section, in Square Inches
Two angles 6 in. × 6 in. × ½ in.;	11.50 − 2 × 2 × .50 =	9.50
One plate 16 in. × ½ in.	8.0 − 2 × .5 =	7.00
One plate 16 in. × $\frac{7}{16}$ in.	7.0 − 2 × .4375 =	6.125
One plate 16 in. × $\frac{7}{16}$ in.	7.0 − 2 × .4375 =	6.125
	Total net area,	28.75

38. In the foregoing design, h_g was assumed. The location of the center of gravity of each flange and the distance between the two centers of gravity will now be computed, and the design of the flanges altered if necessary. For the center of gravity of the lower flange, the statical moment for each angle will be

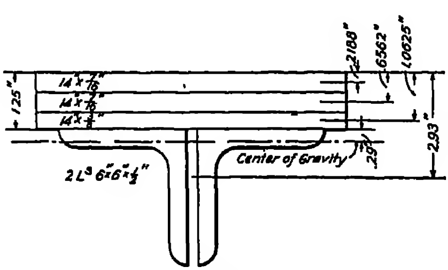


Fig. 11

taken equal to the area of net section multiplied by the lever arm of the gross section, as the position of the center of gravity is practically the same for both sections. The position of the center of gravity of the gross section is taken from Table IX. Moments will be taken about the outer edge of the section in each case, the lever arms for the top flange are shown in Fig. 11, those for the bottom flange in Fig. 12.

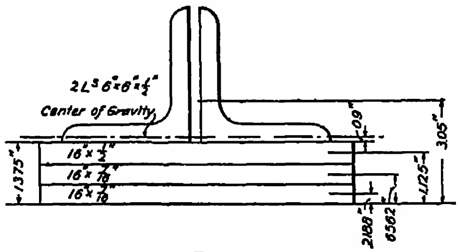


Fig 12

The statical moments for the top flange are as follows:

$$
\begin{array}{lcl}
6.125 \times .2188 & = & 1.340 \\
6.125 \times .6562 & = & 4.019 \\
5.25 \times 1.0625 & = & 5.578 \\
\underline{11.5} \times 2.93 & = & \underline{33.695} \\
29.0 & & 44.632
\end{array}
$$

The distance of the center of gravity of the top flange is, therefore, 44.632 − 29 = 1 54 inches from the outside of the section. As the plates have a total thickness of $\frac{7}{16} + \frac{7}{16} + \frac{3}{8}$ = 1.25 inches, the center of gravity of the top flange is 1.54 − 1.25 = 29 inch below the back of the angles.

The statical moments for the bottom flange are as follows:

6.125	× .2188	=	1.340
6.125	× .6562	=	4.019
7.0	× 1.125	=	7.875
9.50	× 3.05	=	28.975
28.75			42.209

The distance of the center of gravity of this flange from the outside edge of the flange is, therefore, 42.209 ÷ 28.75 = 1.468 inches from the outside of the section. As the plates have a total thickness of $\frac{1}{2} + \frac{7}{16} + \frac{7}{16}$ = 1.375 inches, the center of gravity of the bottom flange is 1.468 − 1.375 = .09 inch from the back of the angles.

As the distance back to back of the flange angles is 66 25 inches, the distance h_g between the centers of gravity of flanges is 66.25 − .29 − .09 = 65.87 inches. This is very nearly equal to the assumed distance of 66 inches. Substituting this value of h_g in the formula for area, the result is

$$A = \frac{2{,}890{,}200 \times 12}{65.87 \times 16{,}000} - \frac{1}{8} \times \frac{1}{2} \times 66 = 32.91 - 4.12$$
$$= 28.79 \text{ square inches}$$

This is slightly greater than the net area of the trial bottom flange, but the difference is so slight (.04 square inch) that it is inadvisable to make any changes. Had the difference been greater—say, .2 or .3 square inch—it might have been advisable to increase the thickness of one of the flange plates by $\frac{1}{16}$ inch.

It will be seen that the flange plates are wider and thicker in the lower flange than in the upper. They are sometimes made the same width, in which case the lower flange plates must be still thicker, or else more plates must be used. If those in the lower flange are made about 2 inches wider than those in the top, and the same number of plates is

used, it will usually be found that the theoretical lengths of corresponding plates in the two flanges are the same or very nearly so; this condition is very convenient in designing, especially if the same rivet spacing is used in both flanges

Some engineers do not design the top flange, but make it the same size as the lower flange. This gives additional section, and therefore additional strength to the top flange, but is not economical. If this were done in the present case, as the gross area of the lower flange is $11.50 + 8 + 7 + 7 = 33.5$ square inches, and the required gross area of the upper flange is 28.79 square inches, the difference, which is 4.71 square inches, would be wasted in the upper flange. Unless it is stated in the specifications that both flanges must have the same gross area, they should be designed separately.

39. Lengths of Flange Plates.—The flange angles in both flanges and the plate next to the flange angles in the top flange are continued the full length of the girder. The other plates are cut off where they are no longer needed. For this purpose, the areas required at the different sections at which the moments have been computed will be determined, and the curves of flange areas will be plotted. The distances between the centers of gravity of the flanges at sections other than at the center are not known, but they may be assumed for trial equal to that at the center, and corrected later. Using the bending moments found in Art. **32**, the required flange areas, in square inches, are:

At the center,

$$\frac{2{,}890{,}200 \times 12}{65.87 \times 16{,}000} - 4.12 = 32.91 - 4.12 = 28.79$$

At 20 feet from the end,

$$\frac{2{,}734{,}700 \times 12}{65.87 \times 16{,}000} - 4.12 = 31.15 - 4.12 = 27.03$$

At 15 feet from the end,

$$\frac{2{,}391{,}700 \times 12}{65.87 \times 16{,}000} - 4.12 = 27.24 - 4.12 = 23.12$$

At 10 feet from the end,

$$\frac{1,811,400 \times 12}{65.87 \times 16,000} - 4.12 = 20.63 - 4.12 = 16.51$$

At 5 feet from the end,

$$\frac{1,034,700 \times 12}{65.87 \times 16,000} - 4.12 = 11.79 - 4.12 = 7.67$$

Fig. 13 shows the curves of flange areas: AB represents the half span; C, D, E, and F, the sections at 5, 10, 15, and 20 feet from A, respectively, CC', DD', EE', FF', and BB',

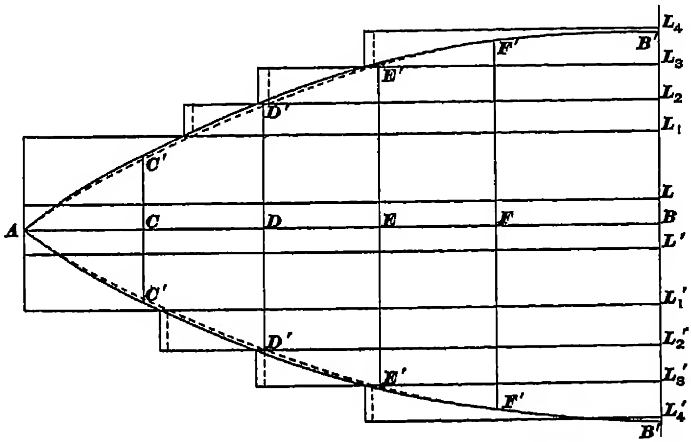


FIG 13

the required areas of the flanges at C, D, E, F, and B, respectively, BL and BL' representing $\frac{th}{8}$, or the portion of web that is included in flange area; LL_1, L_1L_2, L_2L_3, and L_3L_4, the areas of the angles and plates in the upper flange; and $L'L_1'$, $L_1'L_2'$, $L_2'L_3'$, and $L_3'L_4'$, the areas of the angles and plates in the lower flange. Dotted curves are then drawn through A, C', D', E', F', and B'. Drawing lines through L, L_1, L_2, etc., and noting where they intersect the curves, the points at which the plates are no longer required are determined. At C, no plates are required; at D, one plate on each flange is required; at E, two plates on each flange are required; and at F and B, three plates on each flange are required. At F and B there is no need to revise the flange

area, as the entire section is required. In some cases, as at E, the end of a plate is just included in the section, but, as the plate at that point does not carry much stress, it should not be counted as part of the flange area in calculating the distance between the centers of gravity of the flanges.

The actual location of the center of gravity at each section can be calculated in the same way as in Art. **38** for the entire flange, it is not necessary to repeat the numerical steps. They are, for the top flange, .64 inch at E, 1.09 inches at D, 1.68 inches at C, below the backs of the flange angles; and for the bottom flange, .43 inch at E, .86 inch at D, and 1.68 inches at C, above the backs of the flange angles. The distances between the centers of gravity of the flanges are:

$$\begin{aligned}\text{At } E,\ 66.25 - .64 - .43 &= 65.18 \text{ inches}\\ \text{At } D,\ 66.25 - 1.09 - .86 &= 64.30 \text{ inches}\\ \text{At } C,\ 66.25 - 1.68 - 1.68 &= 62.89 \text{ inches}\end{aligned}$$

The revised flange areas, in square inches, are, therefore, as follows:

At 15 feet from the end,

$$\frac{2{,}391{,}700 \times 12}{65.18 \times 16{,}000} - 4.12 = 27.52 - 4.12 = 23.40$$

At 10 feet from the end,

$$\frac{1{,}811{,}400 \times 12}{64.30 \times 16{,}000} - 4.12 = 21.13 - 4.12 = 17.01$$

At 5 feet from the end,

$$\frac{1{,}034{,}700 \times 12}{62.89 \times 16{,}000} - 4.12 = 12.34 - 4.12 = 8.22$$

The curve of flange areas may now be corrected by plotting these values at C, D, and E, drawing the curves shown in full lines, and locating the theoretical ends of the flange plates. The distance scaled from the theoretical end of a plate to the center line is one-half the length of plate; in this case, the theoretical lengths of plates in the top flange are approximately 25, 34.5, and 40.5 feet, and in the bottom flange, 25.5, 34.75, and 43 feet. According to $B.$ $S.$, Art. **60**, each plate

shall extend 12 inches at each end beyond the theoretical end; this will increase the length of each plate by 2 feet. The first plate in the top flange and all the angles will continue the full length of the girder, that is, 55 feet 10 inches. The flanges are then made up as follows:

TOP FLANGE

Two angles 6 in. × 6 in. × $\frac{1}{2}$ in. × 55 ft. 10 in
One plate 14 in. × $\frac{3}{8}$ in. × 55 ft. 10 in.
One plate 14 in. × $\frac{7}{16}$ in. × 36 ft. 6 in.
One plate 14 in. × $\frac{7}{16}$ in. × 27 ft.

BOTTOM FLANGE

Two angles 6 in. × 6 in. × $\frac{1}{2}$ in. × 55 ft. 10 in.
One plate 16 in. × $\frac{1}{2}$ in. × 45 ft.
One plate 16 in. × $\frac{7}{16}$ in. × 36 ft. 9 in.
One plate 16 in. × $\frac{7}{16}$ in. × 27 ft. 6 in.

The actual lengths of the plates will probably be slightly different from the lengths given, the difference being due to rivet spacing in the flanges.

40. Splices.—As no plate or angle is longer than 70 feet, it is unnecessary to splice any flange member (*B. S.*, Art. **61**). Consulting Table V, it is found that the longest plate 66 in. × $\frac{1}{2}$ in. that it is possible to get is 34 feet long; it is therefore necessary to splice the web. It will be spliced at the center, making each half 27 feet 11 inches long, nearly. The size of the splice plates will first be determined. Consulting Table XI, it is found that for a 6″ × 6″ × $\frac{3}{8}$″ angle the nominal and actual widths are equal; then, the actual size of the leg of a 6″ × 6″ × $\frac{1}{2}$″ angle is $6\frac{1}{8}$ inches. As the distance back to back of the flange angles is $66\frac{1}{4}$ inches, the clear distance between the vertical legs is $66\frac{1}{4} - 6\frac{1}{8} - 6\frac{1}{8} = 54$ inches. Allowing $\frac{1}{8}$ inch clearance at top and bottom leaves $53\frac{3}{4}$ inches as the height of the splice plate. According to *B. S.*, Art. **56**, each splice plate shall have a sectional area equal to 75 per cent. that of the web. In the present case, that of the web is 33 square inches; then, the area of each plate must be .75 × 33 = 24.75 square inches. As the plates are

53¾ inches deep, the required thickness is 24.75 ÷ 53.75 = .46 inch. The nearest standard thickness is ½ inch; then, each splice plate will be 53¾ in. × ½ in. A spacing of rivets will now be assumed; if it gives sufficient resistance, it will be used; if not, the number of rivets will be increased. The spacing shown in Fig. 14 will be assumed.

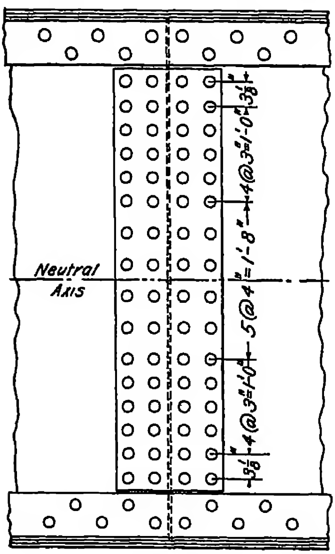


FIG. 14

The rivets are in double shear and in bearing on a ½-inch web-plate. It has already been found (Art. 35) that the latter gives the smaller value for a ⅞-inch rivet; this value is 9,630 pounds. As the distances of the rivets from the neutral axis are 2, 6, 10, 13, 16, 19, 22, and 25⅛ inches, and as there are four rivets at each of these distances, the moment of resistance of the rivets is, according to the formula given in *Design of Plate Girders*, Part 1,

$$\frac{9,630 \times 4 \times (2^2 + 6^2 + 10^2 + 13^2 + 16^2 + 19^2 + 22^2 + 25.125^2)}{25.125}$$
$$= 3,130,000 \text{ inch-pounds}$$

The moment that the web can bear is given by the formula $M = \frac{s t h^2}{8}$ (*Design of Plate Girders*, Part 1). In the present case, the value of s for bending stress is 16,000 pounds per square inch, t = ½ inch, and h = 66 inches; then,

$$M = \frac{16,000 \times .5 \times 66^2}{8} = 4,356,000 \text{ inch-pounds}$$

Since this is greater than the resisting moment that has been found, it is necessary to use more rivets. As the rivets farthest from the neutral axis are the most effective, we shall begin another row outside of the first two rows, and first compute the moment of resistance of a few rivets near

the flanges. Let us first try two rivets at the top and two at the bottom, at distances of 25.125 and 22 inches from the neutral axis. The moment of resistance of these four rivets is

$$2 \times \frac{9{,}630 \times (22^2 + 25.125^2)}{25.125} = 855{,}000 \text{ inch-pounds,}$$

which, added to that already found, gives 3,130,000 + 855,000 = 3,985,000 inch-pounds. This is still too small. Let us try one more rivet at the top and one at the bottom in the same row, each 19 inches from the neutral axis. The moment of resistance of these two is

$$2 \times \frac{9{,}630 \times 19^2}{25\ 125} = 277{,}000 \text{ inch-pounds,}$$

which, added to that already found, gives 3,985,000 + 277,000 = 4,262,000 inch-pounds. This is still too small. Let us try one more rivet at the top and one more at the bottom in the same row, each 16 inches from the neutral axis. The moment of resistance of these two is

$$2 \times \frac{9{,}630 \times 16^2}{25.125}$$

= 196,000 inch-pounds, which, added to that already found, gives 4,262,000 + 196,000 = 4,458,000 inch-pounds. This is a little larger than the moment of the web, and is, therefore, sufficient. Three splice plates on each side of the web will now be used instead of one, the top and bottom plates having three rows of rivets on each side of the splice and the middle plate two rows. It is necessary to rearrange the spacing of the rivets, so that there will be the proper distance from the edge of each plate to

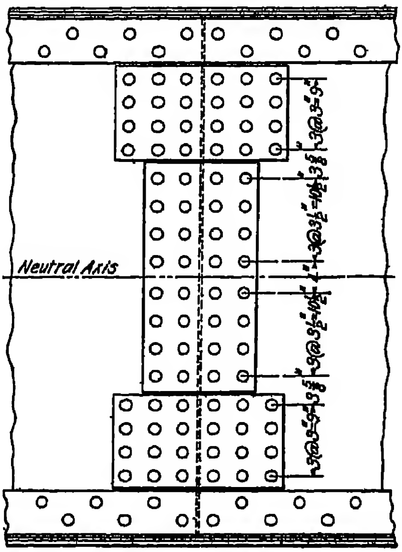


FIG 15

the nearest rivet, and $\frac{1}{8}$ inch clearance between the plates (see *B. S.*, Art. **41**). The spacing represented in Fig. 15 will be used, this changes the location of the rivets, and it is well to recompute the moment of resistance of the rivets in the entire splice That moment is as follows: $M = 9{,}630 \times [4 \times (2^2 + 5.5^2 + 9^2 + 12.5^2) + 6 \times (16.125^2 + 19.125^2 + 22.125^2 + 25.125^2)] - 25.125 = 4{,}433{,}000$ inch-pounds, which is larger than the moment of resistance of the web, and therefore sufficient. It is unnecessary to calculate the moment of resistance of the splice plates; if the plates on each side have an area on a vertical section 75 per cent. that of the web, their moment of resistance will be greater than that of the web.

41. Bearings.—Since the abutments are granite, for which, according to *B. S*, Art. **29**, the allowable intensity of pressure is 500 pounds per square inch, and the end shear, which is equal to the reaction, is 240,600 pounds, the required area of bearing is $240{,}600 - 500 = 481.2$ square inches. In Art **26**, it was stated that the bedplates would be made 22 inches long; the required width is, therefore, $481.2 \div 22 = 21.9$ inches. They will be made 22 inches long.

42. End Stiffeners.—The formula given in *Design of Plate Girders*, Part 1, for the required thickness of end stiffeners is $t' = \frac{R}{n s_b (b - \frac{1}{2})}$. In the present case, $R = 240{,}600$ pounds, and, according to *B. S.*, Art. **29**, the allowable intensity of bearing s_b is 18,000 pounds per square inch. The outstanding legs of the flange angles are 6 inches wide; then, according to *B. S.*, Art. **55**, the stiffeners will be 5 in. × $3\frac{1}{2}$ in. It will first be assumed that there are four stiffeners; this gives

$$t' = \frac{240{,}600}{4 \times 18{,}000 \times (5 - \frac{1}{2})} = .743 \text{ inch}$$

When the required thickness of stiffeners comes out greater than $\frac{5}{8}$ inch, as in this case, it is generally considered advisable to use more stiffeners Using eight gives

$$t' = \frac{240{,}600}{8 \times 18{,}000 \times (5 - \frac{1}{2})} = .371 \text{ inch, say, } \frac{3}{8} \text{ inch}$$

As this thickness is less than $\frac{5}{8}$ inch, it will be adopted, and eight stiffeners 5 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. with reinforcing plates under them will be used The stiffeners will be riveted to the girder with $\frac{7}{8}$-inch rivets. The value in single shear, 6,610 pounds (Table XL), is found to be the smallest value. Then, the number of rivets required to connect each stiffener is

$$\frac{240,600}{8 \times 6,601} = 4\ 55\text{, or, say, 5 rivets}$$

43. Lateral System.—The lateral truss represented in Fig. 16 will be used for both the upper and the lower flange; the end frames will be about 52 feet apart, which makes it possible to use eight panels at 6 feet 6 inches each. The wind load is given in *B. S.*, Art. **27.** The pressure on the lower half of the girder, about 3 feet in depth, is resisted by the lower laterals; the pressure of 50 pounds per square foot, or, in this case, $3 \times 50 = 150$ pounds per linear foot, evidently causes the greatest stresses in the lower lateral truss.

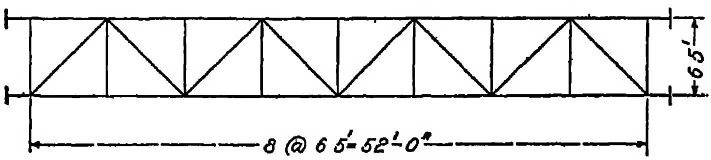


FIG 16

Then, the panel load for the lower lateral truss is $150 \times 6.5 = 975$ pounds. The pressure on the upper half of the girder, and on the rails and ties, say 4 feet in depth, together with the pressure of 300 pounds per linear foot on the train, are resisted by the upper lateral system. The pressure on the train—together with 30 pounds per square foot, or $4 \times 30 = 120$ pounds per linear foot on the girders, ties, and rails—evidently causes greater stresses than 50 pounds per square foot on the girders, ties, and rails alone. The live wind panel load for the upper lateral system is, therefore, $300 \times 6.5 = 1,950$ pounds, and the dead wind panel load, $120 \times 6.5 = 780$ pounds.

44. The end panel of the upper lateral truss will first be considered. The live-load shear is $\frac{1{,}950 \times 7}{2} = 6{,}825$ pounds, and the dead-load shear is $\frac{780 \times 7}{2} = 2{,}730$ pounds The total shear is, therefore, $6{,}825 + 2{,}730 = 9{,}555$ pounds. The panel length is the same as the distance center to center of girders; so the inclination of the laterals is about 45°; csc 45° = 1 414 The direct stress in the diagonal is $9{,}555 \times 1\ 414 = 13{,}510$ pounds, tension when the wind blows in one direction, and compression when in the other direction. According to *B. S.*, Art. **34,** the member must be designed for $13{,}510 + .8 \times 13{,}510 = 24{,}320$ pounds tension and compression Dividing by 16,000 gives $24{,}320 \div 16{,}000 = 1.52$ square inches net section required to resist the tension.

45. According to *B. S.*, Art. **86,** the smallest angle that can be used for lateral bracing is $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. In Table IX the gross area of this angle is given as 2.48 square inches. The number of holes to be deducted depends on the method of riveting the angle to the connection plate. Fig. 17 shows a connection frequently used: the short angle *a* riveted to the main angle at the end is called a **lug angle,** and serves the purpose of transmitting the stress from the leg *b* of the main angle to the connection plates. In practice, the number of rivets connecting the two angles is usually made one more than half the number required to connect the lateral to the connection plate. With rivets spaced as shown, 1 5 holes must be deducted, according to *B. S.*, Art. **33.** As the angle is $\frac{3}{8}$ inch thick, the area of cross-section of one hole is .375, and of 1.5 holes,

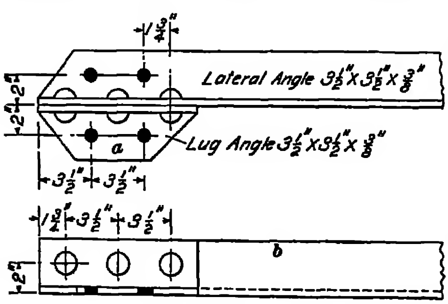


FIG 17

$1.5 \times .375 = .5625$ square inch. Deducting this from 2.48 leaves 1 92 square inches net section. As only 1.52 is required, this angle is large enough so far as tension is concerned.

46. The distance center to center of girders, measured along a diagonal, is $6.5 \times 1.414 = 9.19$ feet $= 110$ inches, nearly. The ends of the laterals are riveted to the connection plates, so that the unsupported length may be taken as the distance between connections, or about 18 inches at each end shorter than the distance between girders, leaving $110 - 2 \times 18 = 74$ inches unsupported. Table IX gives the least radius of gyration as .68 inch, then,

$$\frac{l}{r} = \frac{74}{.68} = 108.82$$

Table XXXV gives the allowable intensity of compressive stress as 9,650 pounds; as the gross area is 2.48 square inches, the strength of the angle is $2.48 \times 9{,}650 = 23{,}930$ pounds. This is very nearly equal to 24,320, the required strength, and so this angle is sufficiently large.

47. As the span under consideration is less than 75 feet long, it will be shipped riveted up complete. That is, the rivets connecting the laterals to the lateral plates will be shop-driven, and, according to *B. S.*, Art. **29**, the values given in Table XL will be used. Table XL gives the value of a $\frac{7}{8}$-inch rivet in single shear as 6,610 pounds; the required number of rivets is then $24{,}320 \div 6{,}610 = 3.7$, or, say, 4 rivets. As the $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle is strong enough in the end panel of the upper lateral system, it is sufficient in all other panels, and so there is no need in this case of making any computations for the other angles.

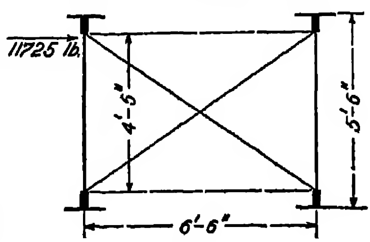


Fig 18

48. The amount of wind pressure that is transmitted to the abutments by the end frames (a diagram of which is shown in Fig. 18) can be found

by multiplying the sum of the live and dead wind loads per linear foot on the upper lateral truss ($300 + 120 = 420$ pounds) by the total length of the girder, which is 55 feet 10 inches, or 55.83 feet As one-half of this load is transmitted by each frame, the amount is $\frac{420 \times 55.8333}{2} = 11{,}725$ pounds. As there are two diagonals, one will be assumed to be out of action when the other is in tension. The height of the frame is about 4.5 feet, the tension in a diagonal is, therefore,

$$11{,}725 \times \frac{\sqrt{4.5^2 + 6.5^2}}{6.5} = 11{,}725 \times \frac{7.89}{6.5} = 14{,}230 \text{ pounds}$$

It has already been found that a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle is more than sufficient for a stress in tension of 24,320 pounds; so that it will be used in this case.